407-444

QUINNIPIAC RIVER BASIN

WOODBRIDGE, CONNECTICUT

LAKE WATROUS DAM
CT 00318

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1978

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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Quinnipiac River Basin
Woodbridge, Connecticut
20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The 1240 feet long dam is a concrete gravity section for the majority of its length with upstream and downstream embankments. The top of the dam is at elevation 228.3, approximately 5 feet above the spillway crest at elevation 223.3, and approximately 51 feet above the old streambed at estimated elevation 174. Based on the visual inspection at the site and past performance history, the dam appears to be in good condition. Based upon the size (intermediate) and hazard classification (high), the Test Flood will be equal to the Probable Maximum Flood.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

I am forwarding to you a copy of the Lake Watrous Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, The New Haven, Water Company, Sargent Drive, New Haven, Connecticut 06506, ATTN: Mr. Jack Reynolds, Superintendent, Source of Supply.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely yours,

JOHN P. CHANDLER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

LAKE WATROUS DAM

CT 00318

QUINNIPIAC RIVER BASIN
WOODBRIDGE, CONNECTICUT

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Inventory Number:	CT 00318
Name of Dam:	LAKE WATROUS DAM
State Located:	CONNECTICUT
County Located:	NEW HAVEN
Town Located:	WOODBRIDGE
Stream:	WEST RIVER
Owner:	NEW HAVEN WATER CO.
Date of Inspection:	JUNE 1, 1978
Inspection Team:	MIKE HORTON
	HECTOR MORENO
	GONZALO CASTRO
	DEAN THOMASSON

The 1240 feet long dam is a concrete gravity section for the majority of its length with upstream and downstream embankments. At the right end of the dam adjacent to the spillway, the concrete section narrows to effectively become a 70 foot long concrete corewall for the surrounding earthen embankments. The top of the dam is at elevation 228.3, approximately 5 feet above the spillway crest at elevation 223.3, and approximately 51 feet above the old streambed at estimated elevation 174. Downstream embankments have a maximum slope inclination of 3 horizontal to 1 vertical. The spillway is a 70 foot wide concrete ogee weir flowing to the spillway channel cut into rock at the right end of the dam. Two 30 inch cast iron pipes pass through the dam; one at elevation 183 is a supply main, and the other at elevation 178, is the low level outlet.

Based on the visual inspection at the site and past performance history, the dam appears to be in good condition. No evidence of structural instability was observed in either the concrete gravity section or the embankment - concrete corewall section of the dam. The downstream earthen embankment was observed to be in good condition, with only a minor surface slump in one area. There are some areas which do, however, require attention.

Based upon our hydraulic computation, the spillway capacity is 2800 cubic feet per second (cfs), which is equivalent to approximately 25 percent of the Test Flood. Based upon the size (Intermediate) and hazard classification (High), in accordance with the Corps guidelines, the Test Flood will be equal to the Probable Maximum Flood (PMF). Peak inflow to the reservoir is calculated to be 12,600 cfs; peak outflow (Test Flood) is 11,400 cfs with the dam overtopped 1.7 feet. The peak failure outflow from the dam breaching would be 193,000 cfs. A breach of the dam would develop a wave which would be 15 feet high downstream at Lake Dawson Dam. Assuming Lake Dawson was operating with a freeboard of 5.5 feet, the resulting 90,000 cfs outflow from Lake Dawson with the Lake Dawson Dam overtopped approximately 9 feet, would cause severe loss of life and property damage in the residential area of Woodbridge approximately 2 miles downstream. Should Lake Dawson Dam breach also under the inflow from Lake Watrous, which is quite possible, damage downstream would be a great deal more extensive.

It is recommended that further hydraulic/hydrologic studies be undertaken to determine the most feasible methods for increasing spillway capacity to an acceptable level. An operation and maintenance plan should be instituted, as described in Section 7.

The above recommendations should be instituted within one year of the owner's receipt of this report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

William O. Doll, P.E.
Chief Engineer
Cahn Engineers, Inc.

This Phase I Inspection Report on Lake Watrous Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Charles G. Tiersch

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

Fred J. Ravens Jr.

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

Saul Cooper

SAUL COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionarly in nature. It would be incorrect to assume that the present condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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APPENDIX

*See Special Note Appendix Section B Availability of Data.

OVERVIEW PHOTO

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS. CAHN ENGINEERS. INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER	NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS	LAKE WATROUS DAM WEST RIVER	WOODBRIDGE CONNECTICUT	DATE 5/31/78 CE # 27 531 GD PAGE ix

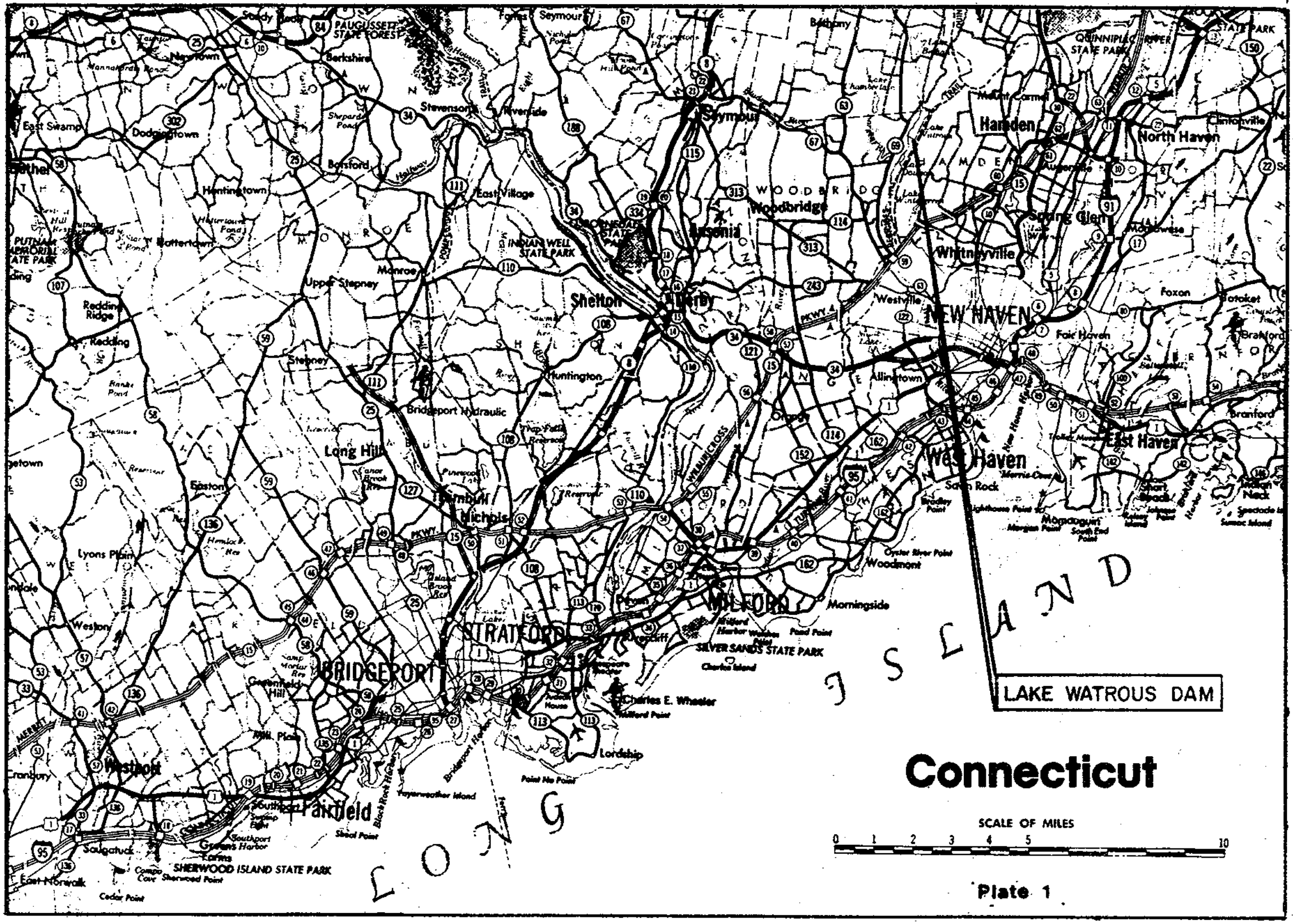
LAKE WATROUS DAM
LONG ISLAND
NEW HAVEN
BRIDGEPORT
STRATFORD
MILFORD
West Haven
Fairfield
Westport
Connecticut
SCALE OF MILES
0 1 2 3 4 5 10

Plate 1

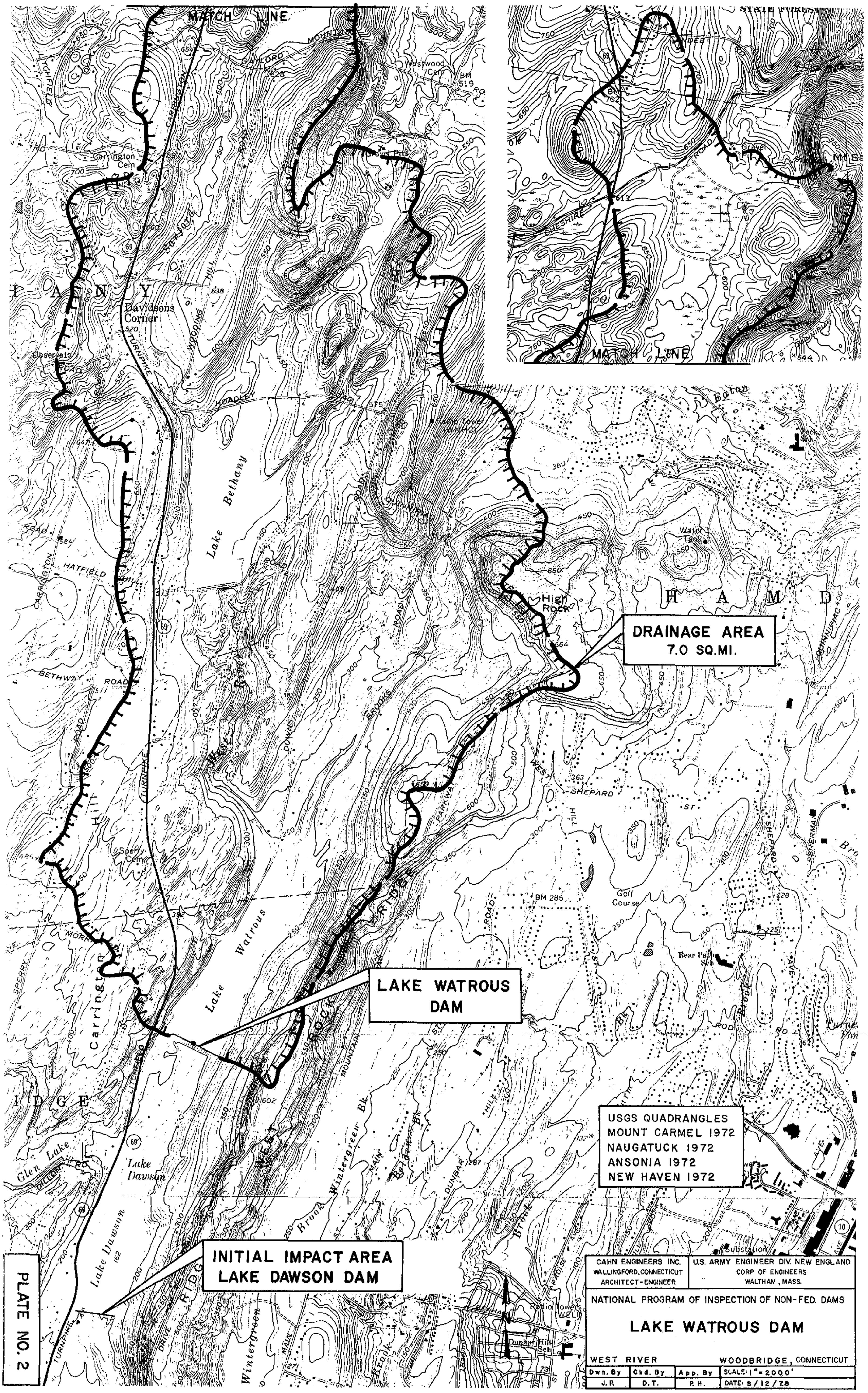

PLATE NO. 2

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ARCHITECT-ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORP OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LAKE WATROUS DAM

WEST RIVER — WOODBRIDGE, CONNECTICUT

Dwn. By	Ckd. By	App. By	SCALE: 1" = 2000'
J.P.	D.T.	P.H.	DATE: 8/12/78

PHASE I INSPECTION REPORT

LAKE WATROUS DAM

SECTION I

PROJECT INFORMATION

1.1 General

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the southwestern portion of the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 26, 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0310 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

(1) Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

(2) Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

(3) To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

(1) Gathering, reviewing and presenting available data as can be obtained from the owners, previous owners, the state and other associated parties.

(2) A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

(3) Computation concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

(4) An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 <u>Description of Project</u>

a. <u>Description of Dam and Appurtenances</u> - The dam is a concrete gravity section approximately 1240 feet long with earthen embankments upstream and downstream. At the west end of the dam adjacent to the spillway, the concrete section narrows down and effectively becomes a concrete corewall for the earthen embankment surrounding it for a length of 70 feet. Maximum embankment slopes downstream are 3 horizontal to 1 vertical. The top of the concrete coping is at elevation 228, approximately 5 feet above the spillway crest at elevation 223, and approximately 51 feet above the streambed at elevation 177.

The spillway is a 70 foot wide concrete ogee section flowing to a spillway channel cut into rock at the right end of the dam. There are two 30 inch cast iron pipes through the dam. One, the low level outlet, is at elevation 178, and the other, the supply main is at elevation 183.

b. <u>Location</u> - The dam is located on West River, in a rural area of the Town of Woodbridge, County of New Haven, State of Connecticut. The dam is shown on the Mount Carmel U.S.G.S. Quadrangle Map having coordinates of longitude W 72° 58' 14" and latitude N 41° 23' 05".

c. <u>Size Classification</u> - INTERMEDIATE - The dam provides storage of 2780 acre feet with the water level at the top of the dam, elevation 228, which is 51 feet above the elevation of the old streambed.

d. <u>Hazard Classification</u> - HIGH (Category I) The dam is located upstream of Lake Dawson Dam, the Wilbur Cross Parkway, and 3 miles upstream of a residential area of Woodbridge.

e. Ownership - New Haven Water Company
Sargent Drive
New Haven, Connecticut 06506
Mr. Joseph Jiskra
Mr. Jack Reynolds
Phone (203) 624-6711

f. Purpose of Dam - Public Water Supply

g. Design and Construction History - The dam was constructed for the New Haven Water Company by C.W. Blakeslee and Sons, Inc. during the period of 1912 to 1915, as engineered by Albert B. Hill, and to our knowledge, has not been modified in the interim period.

h. Normal Operational Procedures - The 30 inch supply main is open at all times and the 30 inch low level outlet is open once a year in the spring.

1.3 Pertinent Data

a. Drainage Area - 7.0 square miles. Rolling, wooded terrain.

b. Discharge at Dam Site - Maximum flood of record - Oct. 16, 1955. Water rose from 2.5 feet below spillway crest to 1.9 feet above spillway crest. Spillway capacity at test flood elevation - 2800 cubic feet per second.

c. Elevations - (Ft. above MSL, U.S.G.S. Datum)

Top Dam:	228.3
Spillway Crest:	223.3
Streambed @ Center Line of Dam:	177±
High Level Intake:	183
Low Level Intake:	179
Outlet Pipe:	178

d. Reservoir

Length of Normal Pool:	4000'
Length of Test Flood Pool:	4000'+

e. Storage

At Elevation 223.3:	2230 acre ft.
At Elevation 228.3: (top of dam)	2800 acre ft.

f.	Reservoir Surface -	At Elevation 223.3:	109 acres
		At Elevation 228.3: (top of dam)	109+ acres
g.	Dam - Type:		Concrete gravity section with upstream & downstream earthen embankments.
	Length:	Dam:	1240 ft.
		Corewall:	70 ft.
	Height:		51 ft.
	Top Width:		10' Minimum-Dam 4' Maximum-Corewall
	Sideslope:		4H to 1V upstream 3H to 1V downstream
	Cutoff:		Founded on rock.

h. Diversion and Regulatory Tunnel - Not Applicable

i.	Spillway -	Type:	Rounded ogee concrete weir
		Length of Weir:	70 ft.
		Crest Elevation:	223.3
		Upstream Channel:	7H to 1V
		Downstream Channel:	30' feet wide typical, natural rock formation.

j. Regulatory Outlets - High Level Intake: Size 30 inch iameter cast iron. Manually operated. At elevation 183. sed as supply main. Open at all times.

Low Level Intake: Size 30 inch diameter cast iron. anually operated once a year in spring. At elevation 178. perational.

SECTION 2: ENGINEERING DATA

2.1 Design

a. Available Data - The available data consists of drawings, records, correspondence and calculations by the State of Connecticut Water Resources Commission, the New Haven Water Company, Joseph W. Cone, Malcolm Pirnie Engineers, and others. The majority of information available pertains to the hydraulic/hydrologic nature of the facility and is included in the Appendix Section B.

b. Design Features - The maps and drawings indicate the design features stated previously herein.

c. Design Data - There were no engineering values, assumptions, test results, or calculations available for the dam construction.

2.2 Construction

a. Available Data - "As-Built" drawings by Albert B. Hill were available and are included in the Appendix Section B. No other construction estimates or reports were available.

b. Construction Considerations - No information was available.

2.3 Operation

a. A representative of the New Haven Water Company stated that the supply main remains open at all times and the low level line is usually opened only once a year in the spring.

2.4 Evaluation

a. Availability - Existing data was provided by the State of Connecticut and the New Haven Water Company. The owner made operations available for visual inspection.

b. Adequacy - The existing data was inadequate to perform a detailed assessment, therefore, the final assessment of the investigation must be based primarily on visual inspection, performance history, and hydraulic/hydrologic assumptions.

c. Validity - A comparison of record data and results of the visual investigations reveals no observable significant discrepencies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 Findings

a. General - The dam is in good condition and requires only some minor maintenance.

b. Dam - The dam consists of a concrete gravity wall with an earth embankment downstream of the concrete wall. The earth embankment is in good condition showing no indication of deformations, sloughing or erosion with the exception of one location where minor sloughing was noted. No seeps were observed through the embankment slope, at the toe or downstream of the dam. The downstream slope of the embankment is covered with well-maintained grass. The drawings indicate a stone drain placed against the downstream face of the dam at the expansion joints connected to an horizontal drain under the downstream embankment. There was no visual evidence of an outlet for the horizontal drain. The upper end of the drain against the downstream face of the concrete wall does not reach the surface of the downstream embankment crest according to the drawings, and thus the presence of the drain could not be visually verified. Minor seepage was observed at horizontal construction joints and expansion joints at various locations in the downstream face of the dam.

c. Appurtenant Structures - The low level outlet structure has concrete walls which are in good condition. At the time of our inspection, a tree had fallen over the outlet structure.

3.2 Evaluation

The visual inspection was sufficient to determine that the condition of the dam and its appurtenant structures appears good with no visual evidence of any stability problem. Seepage observed at expansion joints and horizontal construction joints appeared to be minor resulting only in spalling of the concrete at the joints at the time of our inspection. It was observed that the metal railings, bridge, and protective guard railings are pitted and in need of paint.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Regulating Procedure

The only regulating procedures employed consist of leaving the supply main open at all times to maintain the downstream water supply, and opening the low level outlet once a year, usually in the spring as a maintenance check.

4.2 Maintenance of Dam

The downstream slope of the earth embankment is covered with a well maintained grass cover. The metal railings, bridge and protective guard rails are pitted and in need of paint.

4.3 Maintenance and Operating Facilities

Maintenance of the facility is on an as needed basis as observed during visits to the dam by representatives of the owner. No formal procedures are known to exist.

4.4 Description of any Warning System In Effect

No formal warning system is in effect. Emergencies are reported to the New Haven Water Company office.

4.5 Evaluation

A program of formal operation and maintenance procedures, including thorough, complete documentation of all procedures, should be instituted. A formal warning system should be developed to warn the downstream population in case of emergency.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

a. Design Data - No computations could be found for the dam construction.

b. Experience Data - During the August and October 1955 floods, the maximum water over the spillway was on October 16, 1955, when the water level rose from 2.5 feet below the spillway to 1.9 feet above the spillway.

c. Visual Observations - On the date of our inspection, the spillway was clear and unobstructed. The spillway is wide and appears that it would not be blocked unless debris was retained by the bridge spanning the spillway.

d. Overtopping Potential - The test flood for this high hazard intermediate size dam is equivalent to the Probable Maximum Flood (PMF) of 11,400 cubic feet per second (cfs).

Based upon our hydraulic computations, the spillway capacity is 2800 cfs (Appendix D-9). Based upon "Preliminary Guidance for Estimating Probable Discharges" dated March 1978, peak inflow to the reservoir is 12,600 cfs (Appendix D-8); peak outflow (Test Flood) is 11,400 cfs with the dam overtopped 1.7 feet (Appendix D-12).

e. Spillway Adequacy - The spillway will pass 25% of the Test Flood at the top of dam, elevation 228.

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations - Visual observations do not indicate any apparent stability problem. The masonry dam shows no signs of instability and the earth embankment adjacent to the dam is undisturbed. Inspection of the concrete corewall at the right end of the dam does not indicate any erosion or deterioration.

b. Design and Construction Data - The design and construction reflected in the "As-Built" drawings indicate that the concrete wall foundation is at least 10 ft and as much as 50 ft into either phyllite or sandstone bedrock. The stability of the concrete wall and the downstream earth embankment cannot be formally evaluated with the available information. Such an evaluation depends, for example, on the character of the natural soil and bedrock in which the concrete wall is embedded and the backfilling procedure used against the downstream face of the concrete wall. The available data does not indicate that a seepage or stability analysis has ever been made. Therefore, the determination of dam stability must be based solely on visual inspections and the past performance record of the dam.

c. Operating Records - The dam was built in 1914, and to our knowledge, there have been no indications of instability since construction.

d. Post Construction Changes - There are no post-construction changes indicated in the available records.

e. Seismic Stability - This dam is in Seismic Zone 1 and hence does not have to be evaluated for seismic stability, according to the Recommended Guidelines.

SECTION: 7 ASSESSMENT, RECOMMENDATIONS, & REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition - Based upon the visual inspection at the site and past performance, the dam is judged to be in good condition. No evidence of structural instability was observed in the concrete gravity section, the embankment corewall at the right end of the dam, or in the embankment itself. The embankment is generally in good condition with only one minor area of sloughing observed. There are some areas requiring attention, such as the amount of spillway capacity presently available, the lack of a formal warning system, and the possibility of spillway blockage due to debris at times of high water levels.

Based upon our hydraulics computations, the spillway capacity is 2800 cubic feet per second (cfs), which is equivalent to approximately 25 percent of the Test Flood. Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March 1978, peak inflow to the reservoir is 12,600 cfs; peak outflow (Test Flood) is 11,400 cfs with the dam overtopped 1.7 feet.

Utilizing the April 1978 "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam would be 193,000 cfs. The average stage downstream at Lake Dawson would be 25 feet. Lake Dawson Dam would be overtopped by approximately 15 feet and would most likely breach. Even should Lake Dawson Dam not breach, the 15 foot overtopping would cause severe loss of life and damage to property downstream in residential Woodbridge.

b. Adequacy of Information - A review of the "As-Built" drawings of the structure indicated that the drawings, verified and supplemented as required (see Section 6.1.b), could be used for a detailed structural analysis of the dam should it become necessary. This evaluation of the dam has been based only on the visual inspection and the "As-Built" drawings.

c. Urgency - The actions presented in Sections 7.2 and 7.3 should be implemented within the time frames indicated in each section.

d. <u>Need for Additional Information</u> - There is a need for additional information as noted in Section 7.2.

7.2 <u>Recommendations</u>

The following recommendation should be instituted within one year of the owner's receipt of this Phase I Inspection Report.

1. Based upon the rough computation in Appendix D, the dam spillway capacity will be exceeded by the test flood. More sophisticated flood routing should be undertaken by hydrologists/hydraulics engineers to refine the test flood figures. A study should be undertaken and recommendations made to increase the spillway capacity to an acceptable level based upon the refined test flood figures.

7.3 <u>Remedial Measures</u>

a. <u>Alternatives</u> - This study has identified no practical alternatives to the above recommendations.

b. <u>Operation and Maintenance Procedures</u> - The following measures should be undertaken within one year of the owner's receipt of this report, and continued on a regular basis where applicable.

1. Expansion joints, and any horizontal construction joints which are presently leaking, should be cleaned out, spalled concrete repaired, and the joints caulked. At present, seepage at horizontal construction joints appears to be minor, but if not repaired, concrete deterioration will progress and seepage will increase.

2. Fallen trees and any other debris should be removed from the low level outlet structure. Any trees in the area which might possibly block the outlet structure in the future should also be removed.

3. Areas of minor sloughing on the downstream slope face should be observed periodically to ascertain that no further sloughing is occurring. Should the problem become worse, areas of the slope subject to the sloughing should be repaired with angular stone to increase slope stability.

4. Metal railings, protective guard rails, and the bridge structure spanning the spillway are pitted and should be painted.

5. During the course of this study, it was brought to our attention that the New Haven Water Company instituted a yearly program for inspection of all their dams, including Lake Watrous Dam, by a consultant competent in the field of dam inspection. This program, in effect for two years, is commendable and should be continued in the future.

6. A formal program of operation and maintenance procedures should be instituted, and fully documented to provide accurate records for future reference.

7. Round-the-clock surveillance should be provided by the owner during periods of unusually heavy precipitation. The owner should develop a formal warning system with local officials for alerting downstream residents in case of emergency.

8. As the bottom of the bridge spanning the spillway is at the same elevation as the top of the dam, consideration should be given to raising the bridge and/or providing a log boom to prevent the blockage of the spillway due to floating debris at times of high water levels.

APPENDIX

SECTION A: VISUAL OBSERVATIONS

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Lake Watrous

DATE: June 1, 1978

TIME:

WEATHER: Clear, Sunny

W.S. ELEV. 220 **U.S.** 211 **DN.S**

PARTY:	**INITIALS:**	**DISCIPLINE:**
1. Mike Horton	MH	Structural
2. Hector Moreno	HM	Hydraulic
3. Gonzalo Castro	GC	Geotechnical
4. Dean Thomasson	DT	Party Chief
5.		
6.		

PROJECT FEATURE	**INSPECTED BY**	**REMARKS**
1. Concrete and Earth Dam Embankment	GC	
2. Spillway-Approach, Channel, Weir, Discharge Channel	GC/MH	
3. Outlet Works-Inlet Channel and Inlet Structure	GC	
4. Outlet Works-Outlet Channel and Outler Structure	GC/MH	
5. Concrete Dam Embankment	MH	
6. Outlet Works-Control Tower, Operating House, Gate Shafts	MH	
7. Reservoir	DT	
8. Operation and Maintenance	DT	
9. Safety and Performance Instrumentation	DT	
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Concrete and Earth Dam Embankment

AREA EVALUATED	BY	CONDITION
Concrete Structure		
Crest Elevation		
Current Pool Elevation		
Maximum Impoundment to Date		
General Condition of Concrete Surfaces		
Condition of Joints	MH	Vertical joints at monoliths-spalling, some seepage.
Spalling		Horizontal construction joints-spalling, staining, efflorsence.
Visible Reinforcing		
Rusting or Staining of Concrete	MH	Yes.
Any Seepage of Efflorescence	MH	Yes, some.
Joint Alignment	MH	Good.
Cracking		
Rusting or Corrosion of Steel	MH	None observed.
Erosion or Cavitation	MH	None observed.
Alignment of Monoliths	MH	Good.
Numbering of Monoliths		
Differential Settlement	GC	None observed.
Condition of Structure Foundation		
Structure Additions		
Differential Settlement		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Concrete and Earth Dam Embankment

AREA EVALUATED	BY	CONDITION
<u>Earth Fill</u>		(Earth fill downstream of concrete structure as per drawings)
Surface Cracks	GC	None observed.
Lateral Movement	GC	None observed.
Vertical Alignment	GC	No misalignment apparent.
Horizontal Alignment	GC	No misalignment apparent.
Condition at Abutment and at Concrete Structures	GC	Good.
Indications of Movement of Structural Items on Slopes	GC	None observed.
Trespassing on Slopes	GC	None apparent.
Sloughing or Erosion of Slopes or Abutments	GC	Minor sloughing observed at one location.
Rock Slope Protection-Riprap Failures		No riprap, upstream face is concrete.
Unusual Movement or Cracking at or near Toes	GC	None observed.
Unusual Embankment or Downstream Seepage	GC	None observed.
Piping or Boils	GC	None observed.
Foundation Drainage Features	GC	None according to drawings.
Toe Drains	GC	Chimney drain behind construction joints and horizontal drain at stream bed. Outlet could not be located.
Instrumentation System		
Condition at Joint in Concrete Section		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Spillway-Approach, Channel, Weir, Discharge Channel

AREA EVALUATED	BY	CONDITION
a. Approach Channel	GC	Not observed, reservoir full
General Condition		
Loose Rock Overhanging Channel		
Trees Overhanging Channel		
Floor of Approach Channel		
b. Weir and Training or Sidewalls		
General Condition of Concrete	MH	Very good.
Rust or Staining	MH	None.
Spalling	MH	None.
Any Visible Reinforcing	MH	None
Any Seepage or Efflorescence	MH	None.
Drain Holes		
c. Discharge Channel		
General Condition	GC/MH	Good.
Loose Rock Overhanging Channel	GC	None of any significance observed.
Trees Overhanging Channel	GC	None observed.
Floor of Channel	GC	Bedrock covered with loose boulders within a few hundred feet of spillway, gravelly bottom further, D.S.
Other Obstructions	GC	None observed.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Outlet Works-Inlet Channel & Inlet Structure

AREA EVALUATED	BY	CONDITION
a. Approach Channel	GC	Not observed, reservoir full.
Slope Conditions		
Bottom Conditions		
Rock Slides or Falls		
Log Boom		
Debris		
Condition of Concrete Lining		
Drains or Weep Holes		
b. Intake Structure		
Condition of Concrete		
Stop Logs and Slots		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Outlet Works-Outlet Structure and Outlet Channel *

AREA EVALUATED	BY	CONDITION
General Condition of Concrete	MH	Good.
Rust or Staining		
Spalling		
Erosion or Cavitation		
Visible Reinforcing	MH	None.
Any Seepage or Efflorescence	MH	None.
Condition at Joints		
Drain Holes	GC	None observed.
Channel	GC	Stone walls in good condition.
Loose Rock or Trees Overhanging Channel	GC	Loose trees in discharge channel at outlet and at intersection with spill-way channel.
Condition of Discharge Channel	GC	Good.

*Only blowoff outlet discharges into outlet channel and West River.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Outlet Works-Control Tower, Operating House, Gate Shafts

AREA EVALUATED	BY	CONDITION
a. Concrete and Structural		
General Condition	MH	Good.
Condition of Joints	MH	Good.
Spalling	MH	None.
Visible Reinforcing	MH	None.
Rusting or Staining of Concrete	MH	None.
Any Seepage or Efflorescence	MH	None.
Joint Alignment		
Unusual Seepage or Leaks in Gate Chamber		
Cracks	MH	None.
Rusting or Corrosion of Steel	MH	None.
b. Mechanical and Electrical		
Air Vents		
Float Wells		
Crane Hoist		
Elevator		
Hydraulic System		
Service Gates		
Emergency Gates		
Lighting Protection System		
Emergency Power System		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Reservior

AREA EVALUATED	BY	CONDITION
Shoreline	DT	A road follows the shoreline around the reservoir.
Sedimentation	DT	No problem observed.
Potential Upstream Hazard Areas	DT	None.
Watershed Alteration-Runoff Potential	DT	None.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Operations and Maintenance

AREA EVALUATED	BY	CONDITION
a. Reservoir Regulation Plan		
Normal Conditions	DT	Supply main open constantly; Blowoff open once a year in the spring.
Emergency Plans	DT	No other methods to release water.
Warning System	DT	Call New Haven Water Company Office.
b. Maintenance (Type) (Regularity)		
Dam	DT	As needed.
Spillway	DT	As needed.
Outlet Works	DT	As needed.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Watrous **DATE** June 1, 1978

PROJECT FEATURE Safety and Performance Instrumentation

AREA EVALUATED	BY	CONDITION
Headwater and Tailwater Gages	DT	Headwater gage at spillway.
Horizontal and Vertical Alignment Instrumentation (Concrete Structures)	DT	None.
Horizontal and Vertical Movement, Consolidation, and Pore-Water Pressure Instrumentation (Embankment Structures)	DT	None.
Uplift Instrumentation	DT	None.
Drainage System Instrumentation	DT	None.
Seismic Instrumentation	DT	None.

APPENDIX
SECTION B: EXISTING DATA

SPECIAL NOTE

SECTION B

AVAILABILITY OF DATA

The correspondence listed in the summary of Contents and the plans listed in the Table of Contents, Appendix Section B, are included in the master copy of this report, which is on file at the office of the Army Corps of Engineers, New England Division, in Waltham, Massachusetts.

Only the following correspondence is included in this report.

Date	To	From	Subject	Page
June 26, 1965	New Haven Company	Joseph W. Cone	Report concerning dams owned by New Haven Water Company.	B-16
Aug. 2, 1967	New Haven Company	Malcolm Pirnie Engineers	Investigation of the effect of a flood produced by the Maximum Possible Storm on spillways of West River System.	B-46

SECTION B: EXISTING DATA

SUMMARY OF CONTENTS

DATE	TO	FROM	SUBJECT	PAGE
Original Date March 1,1911 Latest Entry 1969	New Haven Water Company	Albert B. Hill[2]	Reservoir Capacities West River System	B-63
Aug. 1974	Files	New Haven Water Company[2]	Watrous Dam Data Sheet	B-65

[1]Obtained from State of Connecticut Water Resources Commission

[2]Obtained from New Haven Water Company

[3]Hydraulic/Hydrologic Data and Spillway Sections contained in Joseph W. Cone's report, which are on file and available at the New Haven Water Co. office were not included due to poor reproduction quality.

1965

REPORT

CONCERNING DAMS

Owned by

NEW HAVEN WATER CO.

BETHANY

WATROUS

CHAMBERLAIN

GLEN

DAWSON

on the

WEST & SARGENT RIVERS

J. W. Cone P.E.
June 1965

INDEX

Part I

Part II

NOTE: Maps, graphs, etc., are in separate folder.

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ICENSE 4755
CUT REGISTRATION 4

JOSEPH W. CONE
CIVIL ENGINEER
124 HAVEMEYER PLACE
GREENWICH, CONNECTICUT
06830

TELEPHONE
TOWNSEND 9-2152

June 26, 1965

Mr. William P. Sander
Water Resources Commission
State Office Building
Hartford 15, Conn.

Re: Dams #35 - 1 to 5
New Haven Water Co.

Dear Mr. Sander:

First, I apologize for not completing this assignment more promptly; reasons being that a low quality virus for over a month left me with no pep mentally or physically, and delays in obtaining certain plans and information.

The assignment was- "we would like to know the present condition of these dams" - Bethany - Watrous - Dawson on West River and Chamberlain - Glen on Sargent River, a tributory to West River above Dawson Dam.

In my opinion, the "condition" of these dams is good as regards masonry of the three masonry gravity dams and the upkeep of two earth embankment dams.

But as regard to whether or not the dams are safe, <u>particularly as regard spillway capacity</u>, my opinion is as follows:

35-1 <u>Bethany</u> Spillway is inadequate. However a thin sheet over a length of 990' will do comparatively little damage except to highway. The gravity section is safe.

35-2 Watrous Generally same remarks as for Bethany.

35-3 Chamberlain Spillway is adequate in every respect as is the dam. It is reassuring to find a spillway that will carry 1525 cfs per sq. mi. on 4.1 sq. mi. Note Items #26 & 28 on Data Sheet.

35-4 Glen Spillway is nowhere near adequate. In fact, Oct. '55 flood nearly overtopped earth section at left or east abutment. Section of dam is safe.

Right abutment should be raised to protect highway.

Left abutment should be investigated:-

(a) To determine whether or not there is a core wall.

(b) Possibility of emergency spillway or fuse plug.

(c) Note Items #26 & 28 on Data Sheet.

35-5 Dawson Present spillway is entirely inadequate to carry probable floods of the present and future. In fact, the dam would have been overtopped if certain saving factors had not been present in Oct. 1955.

(a) Not an excessive rainfall, only about R of 50 yr. (Compare with precipitation graphs)

(b) Several of reservoirs were below FL (See data notes by Navaro which you have)

(c) Flood Q '55 at Dawson of about 2100 cfs has an R value 3.8 (2100 ÷ 560) equivalent to 120 yr on old Conn. curve and 55 yr on revised 1965 curve. (See graph PL 13)

Items #26 & 28 on Data Sheet are particularly illuminating.

It does not need a lively imagination to visualize what would happen to Westville and New Haven if Dawson should be overtopped; Norwich failure would be peanuts comparatively.

A brief discussion of pertinent data and situations follows. Also there are prints of sections of dams, precipitation graphs and various other graphs that I used or are pertinent to this investigation for general information or checking purposes.

Please excuse the informality and crudness of the matter submitted, the objective being to reduce costs to the minimum.

I would observe that Mr. Navaro, Mr. Ferris and Mr. Reynolds of the New Haven Water Co. were most cooperative as was Mr. Thomas of the U.S. Geological Survey.

My recommendation is that the New Haven Water Co. be advised that their consulting engineers should investigate the entire system, with particular emphasis on conditions at Glen and Dawson, and submit corrective measures.

Yours very truly,

J W Cone

J. W. Cone

JWC/dr

Enc: Part II
Photos (11)

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WATERSHED

<u>Characteristics</u> Area is very rugged, steep side slopes and steep channels. Channel slopes (S in Conn Formula) are West River 70 and Sargent River 88 feet per mile. Elevations on topo sheet point up steepness of side slopes as much as 400' in 0.25 mile.

Area is rural, cover, mostly wooded at present. However within a few decades there will be more intensive land use. There is evidence of this growth in the Cheshire and other areas. At present in spite of rugged terrain, the shed may be considered "medium to fast" due to cover; by about 2000 AD it will become "fast" and in the future could be "very fast".

<u>Area</u> As scaled from 1:24,000 topo sheets area is 13.35 sq. mi. By data in Water Co's. operation office area is 13.0 sq. mi. Mr. Novaro in his report to Mr. Corbin, April 29, 1963, states area is 13.9 sq. mi.; this I do not understand.

	Water Co.	1:24000
Bethany	3.4	3.7
Watrous	3.2	3.3
Chamberlain	3.9	4.1
Glen	1.7	1.6
Dawson	.8	.65
	13.0	13.35

The Company owns about 8 sq. mi. of the 13.35 sq. mi. However as taxes and population pressures increase, as the area becomes more polluted due to development of areas owned by others, it is reasonable to assume that the Company will sell at least 5 sq. mi. and construct a filtration plant. These considerations explain the predicted increase in mean annual flood of about 40% above present by 2000 AD. (560-795 and C_B 0.85-1.2)

The following quote, from an intensive study by Metcalf and Eddy on Storm Water Control in Westchester County in 1945, is pertinent to this discussion. "Residential development of the area has resulted in peak run-off rates almost twice those of twenty-five or thirty years ago, and if development continues at the same rate for the next twenty-five years, the run-off factor will become 2½ times that of conditions a half century ago". It would seem that the increase of 40% is not fantastic.

PRECIPITATION

Data plates 4 to 9 inclusive were studied and are included to determine whether or not the Oct. 1955 storm in the New Haven area was of very rare occurence.

Since the rain gage at Dawson is not recording, graph PL 5 was produced assuming that storm characteristics would be very similar to New Haven Airport which has a recording gage. Similarly the Westfield, Mass. graph was based on Norfolk, Conn.

Using 24 hr values and PL 9 the following recurrence values were determined.

	24 hr in.	Chance %	R
Base	9.5	1.0	100
Dawson	5.85	2.0	50
Norfolk	11.2	0.6	175
Westfield	18.2	0.2	500
Max possible	27.7	0.1	1000

In connection with this subject on Oct. 9, 1877 there was 9.7" in 10.5 hrs. at White Plains, Westchester County, N.Y.

My conclusion is that precipitation in the New Haven area cannot be termed extraordinary. In the Stamford-Norwalk area R values were about 200 yr and in Greenwich about 75.

If precipitation was not excessive then peak flood flow could not be excessive and should have an R value of less than 100.

I realize full well that some may say that I have no right to assign maximum possible to 1000 yrs. My answer is what possible value can the maximum possible values have unless an occurrence value is stated; if no value then data is worthless. Enquiry has been made to many who should be better versed in this matter than I. No one would stick his neck out. I am not afraid to and have; at least a value of 1000 is on the safe side.

My purpose in this discussion is to point out the fact that if either the Norfolk or Westfield precipitations had occurred on this shed in Oct. '55 the resulting disaster would have been appaling.

FLOOD FLOW 1955

Oct. 1955 To determine flood flow at Dawson it is necessary to know H at peak. To check, if H at peak were known for Glen and Watrous, then flow to Dawson could be estimated reasonably close by adding an allowance for the small watershed of Dawson itself.

In this connection I suggest that values shown on Lake Level forms (those were mailed you recently) should not be used since measurements were taken between 8-9 A.M.

The peak of the Oct. flood in Greenwich was about 1 A.M. Allowing for forward speed of storm then peak at Dawson would be between 2-3 A.M. particularly since watershed is "quick". The time lag of about 6 hours would certainly lower H peaks. I therefore, based on conversations and data furnished, assumed certain H values and computed Q, as shown in the following table:

	H	Q	
Glen	3.5	880	cfs
Watrous	3.0	1160	
Dawson shed est		160	
		2200	" to check
Dawson	4	2050	"

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Assuming 2050 correct than R values are:

$$R = \frac{Q}{MAF} = \frac{2050}{560} = 3.7 \pm$$

Refer to PL 13

		R	
By old Conn Curve	3.7	110 yrs	
" new " "	3.7	50 "	

This agrees reasonably well with precipitation value of 50.

Conclusion is that flow of Oct. 1955 at Dawson may be considered a minor flood that would have been somewhat greater had not several of the reservoirs been below FL. for a total of 215 m.g. as computed by Mr. Novaro.

$Q_M = 9\ A^{2/3}$ vs Conn Formula

This formula and graph (PL 12 A & B) has been used for several years with satisfaction. It checks well with the rational method and is much simplier to use. Although designed for small watersheds, up to about two square miles, it fills the gap with considerable reliability up to about ten miles, the approximate reliable lower limit of the Conn Formula, Geological Survey Circular #365.

A = 13.35 sq. mi. = 8500 Ac

8500		3.92942
		2
	3	7.85884
		2.61961
9		0.95424
3748		3.57385

Q_M = 3750 cfs

9

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$\underline{Q_D = RF \times LF \times FF \times Q_M}$

From PL 12 A factors for R = 500, present conditions and 2000 AD Present Q = 1 x 0.4 x 4.35 x 3750 = 6500 $\frac{0.6}{0.4} = 1.50$

2000 AD 1 x 0.6 x 4.35 x 3750 = 9730

$Q = RC_B AS$

By PL #2 $C_B AS$ = 560-present and 795-2000 AD

By PL 13 R for 500 = 11

Q500 Present = 11 x 560 = 6160

" 2000 AD = 11 x 795 = 8745

Note that results are remarkably close, perhaps by coincidence.

	$9 A^{2/3}$	C_B		Conn	C_B	
Present	6500	0.4	150%	6160	0.85	140%
2000 AD	9730	0.6		8745	1.2	

Had basin coefficients (C_B) been selected to obtain the same percent increase in the land use factor, results for 2000 AD would have been 9730 vs 9240.

In any case $Q = 9 A^{2/3}$ provides a reliable check on Conn. Formula, up to about 10 sq. mi., and fills the no-man's gap.

SPILLWAY CAPACITY

cfs. & sq. ft. per sq. mi.

Dam	Type	Q / sq.mi.	cfs	sq.ft.
(1) Bethany	Gravity	$\frac{1980}{3.7}$	540	80
(2) Watrous	"	$\frac{2660}{7}$	380	50 acc
(3) Chamberlain	Earth	$\frac{6300}{4.1}$	1525	120
(4) Glen	Gravity	$\frac{1120}{5.7}$	195	28 acc
(5) Dawson	Earth	$\frac{2870}{13.35}$	215	30 acc

The units shown in this table, for a watershed with nearly the same characteristics throughout, demonstrate the inconsistency in capacity. It is true that an earth dam should have a greater factor of safety than a gravity masonry dam. This data emphasizes the need for corrective measures particularly at Glen and Dawson.

MAF

Check by Comparison

	Est.	Sq. Mi.	Present MAF	per/S.M.
Willow Brook-Cheshire	1960	9.02	280	31
Wepawaug River-Milford	1962	18.00	690	38
		/27.00	/970	
		13.5	485	
Dawson computed PL.2		13.35	560	42
Sargent " "		5.7	425	75

WILLOW BROOK. Rolling terrain, nowhere near as rugged as West River. On other hand land use is more dense. MAF per sq. mi. should be much less than West River.

WEPAWAUG RIVER. Same remarks as above.

SARGENT RIVER. Very steep. S is 88' per mi.

Note that Willow Brook and Wepawaug River stations have only short term records. The usual experience is that the longer the record period the higher are MAF values.

CONCLUSION is that West River MAF of 560 for present land use conditions is not too high and more likely is too low.

(1) BETHANY

BRIDGE. Rough field measurements were taken believing that the bridge would be a bottleneck rather than the spillway. Sketch plan is shown. Later construction plans were available.

Assuming depth of flow in channel as 3' -

$A = 24.5 \times 3 = 73.5$

$P = 24.5 + 6 = 30.5$

$r = 73.5 \div 30.5 = 2.4 \qquad r^{2/3} = 1.8$

$S = .034 \qquad S^{\frac{1}{2}} = 0.18$

Assuming $n = .0148$

$V = 100 \; r^{2/3} \; S^{1/2}$

$= 100 \times 1.8 \times 0.18 = 32$ sf.

$Q = 73.5 \times 32 = 2350 \pm$ cfs.

SPILLWAY. Rough plan shows total length of spillway as 19' + 61' = 80'. But account of turbulence assume effective L = 75', H max = 4', C = 3.3.

$$Q = 3.3 \times 75 \times 8 = 1980 \text{ cfs.}$$

This Q probably maximum due to backup from bridge and turbulence at channel entrance.

From the above it is shown that the spillway rather than the bridge is the limiting factor to carry estimated Q values - Items 14 & 15 on Data Sheet. It is concluded that the dam will be overtopped in the future, with an H value of about 1.

$$Q = 2 \times 990 \times 1^{3/2} = 2080 \text{ cfs}$$

This with spillway on H = 5' will pass over 4000 cfs.

DAM. The gravity section of cement rubble masonry with reinforced concrete back 4' thick is in good condition.

(2) WATROUS

SPILLWAY. The capacity of this 70' spillway with H = 5' is 2660 cfs., as shown by Item 12 on Data Sheet. This capacity will barely take flood flow from its individual watershed below Bethany under present land use, see Items 14 & 15. In addition there is the added flow from Bethany. Total watershed is 7 sq. mi.

Bethany	3.7
Watrous	3.3
	7.0

DAM. The gravity concrete section is in good condition and is backed up with earth nearly to top of dam.

The dam will be overtopped in the future. Note Data Items #26 & 28.

(3) CHAMBERLAIN

A study of items on the Data Sheet and examination of sketch plan indicate that this earth dam is adequate in every respect. No further comment is required.

(4) GLEN

SPILLWAY. The 40' x 4' spillway has a capacity of about 1120 cfs. The entire watershed including Chamberlain is 5.7 sq. mi. Note Data Items #26 & 28.

Chamberlain	4.1
Glen	1.6
	5.7

The dam was nearly over-topped during the October 1955 flood.

ABUTMENTS. A highway is close to the right or west end of spillway. Upstream training wall in particular should be raised and extended.

At the left or east end of the dam there is an area that is lower than crest of dam. This is indicated under the arrow on the photo of the east bank. As determined by hand level, the area is about six inches below dam crest.

There seems to be no record of a core wall in the area or location of ledge surface. If no wall and ledge rock is low, then there will be end scour sometime in the future that would put an extra burden on Dawson. This condition should be investigated.

FUSE-PLUG. The area appears to be favorable for the needed extra spillway capacity, permanent construction, or fuse-plug type.

DAM. The gravity concrete section is in good condition and in my opinion will not fail.

(5) DAWSON

SPILLWAY. An examination of Data Sheet items and study of plans indicate that the Dawson spillway is entirely inadequate. The Q of 2870 with H of 5' is approximate. The combination of a low broad crested humped weir and spillway characteristics present a complicated hydraulic problem not worthwhile to investigate thoroughly for the purpose of this report.

The spillway and right training wall are shown on photo enclosed. Note that the low portion of the training wall was nearly overtopped in Oct. '55.

Height of water at spillway was 3' below dam crest. There must have been considerable velocity head. Therefore if the weir formula is used H should be about 4'.

SEEPAGE. In the area near trees as shown on enclosed photo there is seepage with "guesstimated" flow of about 9 gals per min. Another seepage flow is farther to the west and at a lower elevation near a small cedar with an estimated flow of about 3 gals per min. Both areas should be watched closely.

It would be worthwhile to install a simple arrangement whereby flow can be determined by stop watch timing to fill a container; this to determine whether or not there is a relation between reservoir level and flow.

I have been informed by Mr. Ferris that most of the trees shown in photo have been removed. Trees were not on the embankment proper but were close enough to present the possibility of root-boil trouble.

EMBANKMENT COVER. The easterly portion of the dam, about one half, had been grazed by sheep. This is an inexpensive method of controlling grass on a 1 on 2 slope. On the other hand sheep are close croppers and tend to destroy root structure, a condition evident at the time. If the dam should be overtopped by a few inches I would anticipate that the sheep cropped area would gully seriously.

Further, particularly during dry weather, grass cover should be kept high to provide shade to hold moisture as much as is possible on the steep 1 on 2 slope, where water-table is low, and to prevent baking all of which weakens root structure.

CONCLUSION. It is my opinion that the situation at Dawson is very serious. If a bad breach should occur the refuge in "An Act of God" would not prevail. In Oct. 1955 if all reservoirs had been full, if twenty-four hour precipitation had been a little more, then it is my opinion that Dawson would have been overtopped.

As stated hereinbefore a comprehensive study of this situation should be begun immediately and proposed corrective measures presented as soon as possible.

GENERAL

It is my understanding that my assignment was not to undertake a complete analysis of all aspects involved, but only to investigate sufficiently to determine if there are situations that should be studied by the Company's consulting engineers. I therefore did not undertake the following:

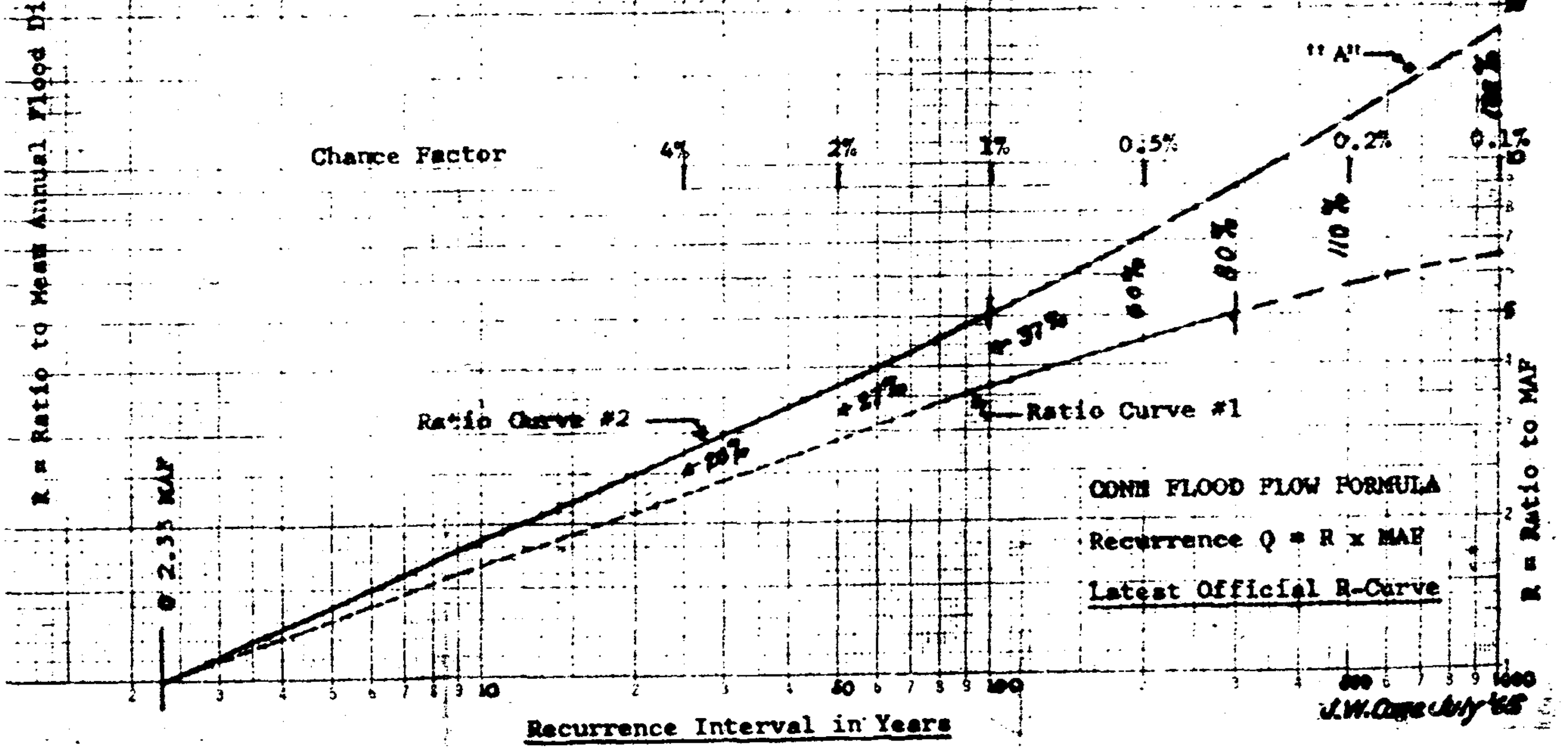
1. Stability analysis of gravity masonry dams. Casual study of plans indicates they are safe; this based on experience.
2. A design flood based on an assumed precipitation was not routed through the several watersheds and reservoirs, considering storage capacity above FL etc. This would have been a tedious study and funds were not available in my contract.
3. In computing the several Q values no credit was given to storage above FL, rather this was considered as an extra factor of safety, to be on the safe side.

Graphs, plans, etc., are bound separately for ease in following the text.

Curve #1 is as shown in G.S. Circular #365, Flood Flow Formula for Connecticut, based on data prior to 1955 Floods.

Curve #2 was derived by M. P. Thomas USGS, after WSP #1671, by A. Rice Green 1964, based on data through water year 1960. This curve has official G.S. approval to 100 yrs. Extension to 1000 yrs. by Thomas using Gumbel's recurrence interval scale.

Point "A" There have been recorded floods 15 MAP during record period of 330 yrs. Assuming that 330 yrs. is a measure of 660 yrs., one-half period before and after observed period, then plot "A" of R=15 at 660. This agrees substantially with #2.



DATA SHEETS

1. Summary of data.
2. Determination of MAF, graphically
3. Watersheds; sketch arrangement
4. Precipitation Oct. '55 New Haven
5. " " " Dawson (devised)
6. " Aug. " Norfolk
7. " " " Westfield (devised)
8. " Maximum Possible
9. " Recurrence 2 to 24 hr.
10. Flood flow graph old.
11. " " " revised.
12.- A Peak Runoff $Q = A^{2/3}$

     B " " "

     C " " "
13. Ratio Curve - Conn Formula
14. Weir Coefficients
15. Plans Bethany (3)
16. " Watrous (1)
17. " Chamberlain (1)
18. " Glen (2)
19. " Dawson (2)

Topo of Watershed 1:24000

COMMENTS re DATA SHEETS

#10 This Flood Flow Curve shown since it shows a curve, dashed line, devised by A.B. Hill about the turn of the century. It was considered a sound base curve at that time when there was a paucity of information as compared to that which became available in more recent years; precipitation and flood flow records, many studies, reports, etc.

#13 The upper curve, shown in red, was plotted by Mr. Mendall P. Thomas with the Geological Survey based on study by A. Rice Green, Water Supply Paper 1671, 1964. Curve has official approval to 100 years; projection to 1000 by Thomas using Gumbel's recurrence interval scale. This is the latest R-curve available.

The purpose of including the other sheets I believe is self-evident.

WAT

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT

MEMORANDUM REPORT TO WATER COMPANY
ON
INVESTIGATION OF THE EFFECTS OF A FLOOD
PRODUCED BY THE MAXIMUM POSSIBLE STORM
ON SPILLWAYS OF WEST RIVER SYSTEM

AUGUST 2, 1967

The effect of the "maximum possible storm" on the West River System is reported in this memorandum.

The "maximum possible storm" employed is defined and quantitatively estimated in U. S. Weather Bureau Hydrometeorological Report No. 33 entitled "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24 and 48 Hours." The report defines the "maximum possible precipitation" as "the critical depth-duration-area rainfall relation for a particular area during various months of the year that would result if conditions during an actual storm in the region were increased to represent the most critical meteorological conditions that are considered probable of occurrence."

As shown on Exhibit 1, the rainfall totals used for the West River System analyses are for durations of 6 and 12 hours on an area of 10 square miles for September -- the most severe month for the vicinity of New Haven, Connecticut. The hourly

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distribution of the total rainfall assumed is according to Figure 4, page 32 of U. S. Department of the Interior publication "Design of Small Dams." The distribution is a comparatively severe one with 50 per cent of the 6 hour total falling within 1 hour.

The sequence in which the hourly totals were arranged is in accordance with the recommendation made on page 50 in "Design of Small Dams." The arrangement of the 12 hourly increments is 11, 9, 7, 5, 3, 1, 2, 4, 6, 8, 10, 12, where the number represents the order of magnitude with the lowest number representing the largest magnitude. This arrangement gives a flood greater than one based on the assumption that the greatest hourly increment of rain occurs during the first hour of a storm

The effective, runoff-producing rainfall was estimated by subtracting 1 inch initial infiltration and 0.1 inch per hour thereafter from the total rainfall.

In order to pass the unusually high flows for the "maximum possible storm," several modifications of both the length and crest height of spillways were tried. Spillway rating curves and stage capacity curves for each of the five reservoirs are shown on Exhibit 2 and Exhibit 3, respectively.

The unit-hydrographs and routing procedures employed are those outlined in our report of January, 1967. Detailed computations are shown on Exhibit 4, pages 1 through 8.

The inflow-outflow curves for each of the reservoirs are shown on Exhibit 5, pages 1 though 3. As no significant storage effect is obtained from Lake Dawson, the outflow hydrograph as shown on Exhibit 5, page 3, will be the same with a spillway 250 feet long.

The "maximum possible" flood outflows at each of the West River reservoirs and the conditions at the Spillways are summarized below:

Dam	Peak Spillway Discharge cfs	Free-Board ft.	Maximum Head (ft.) Over Spillway	Over Dam Crest
Chamberlain	7200	12.0	10.8	-1.2
Glen	9665	9.0*	11.3	+2.3
Bethany	7350	4.25	5.2	+1.0
Watrous	15,400	5.0	7.1	+2.1
Dawson				
80' Spillway	26,260	11.5*	13.8	+2.3
250' Spillway	26,260	11.0*	9.0	-2.0

*Freeboard above proposed new sill elevation

EXHIBIT 1

MAXIMUM POSSIBLE RAINFALL FOR NEW HAVEN, CONNECTICUT

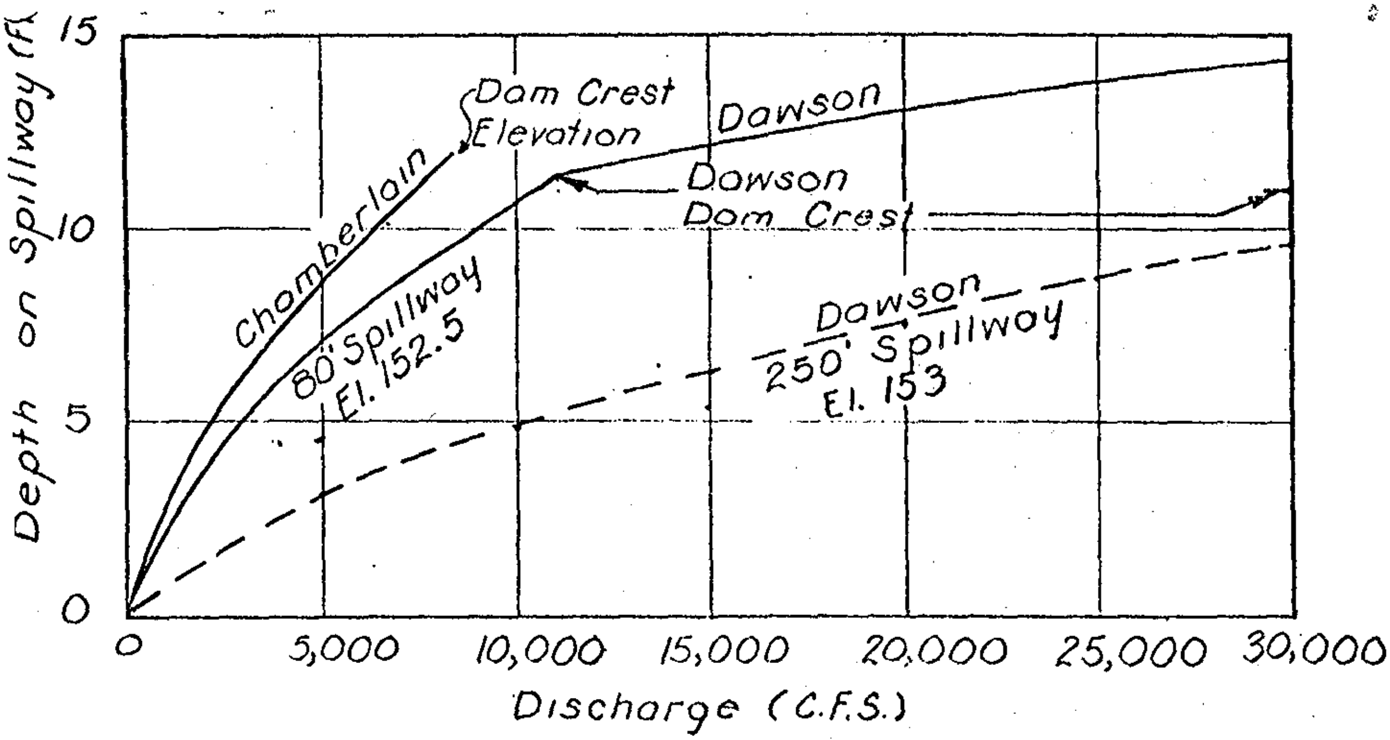
*DURATION OF RAINFALL HOURS	TOTAL RAINFALL INCHES
6	24.2
12	26.4

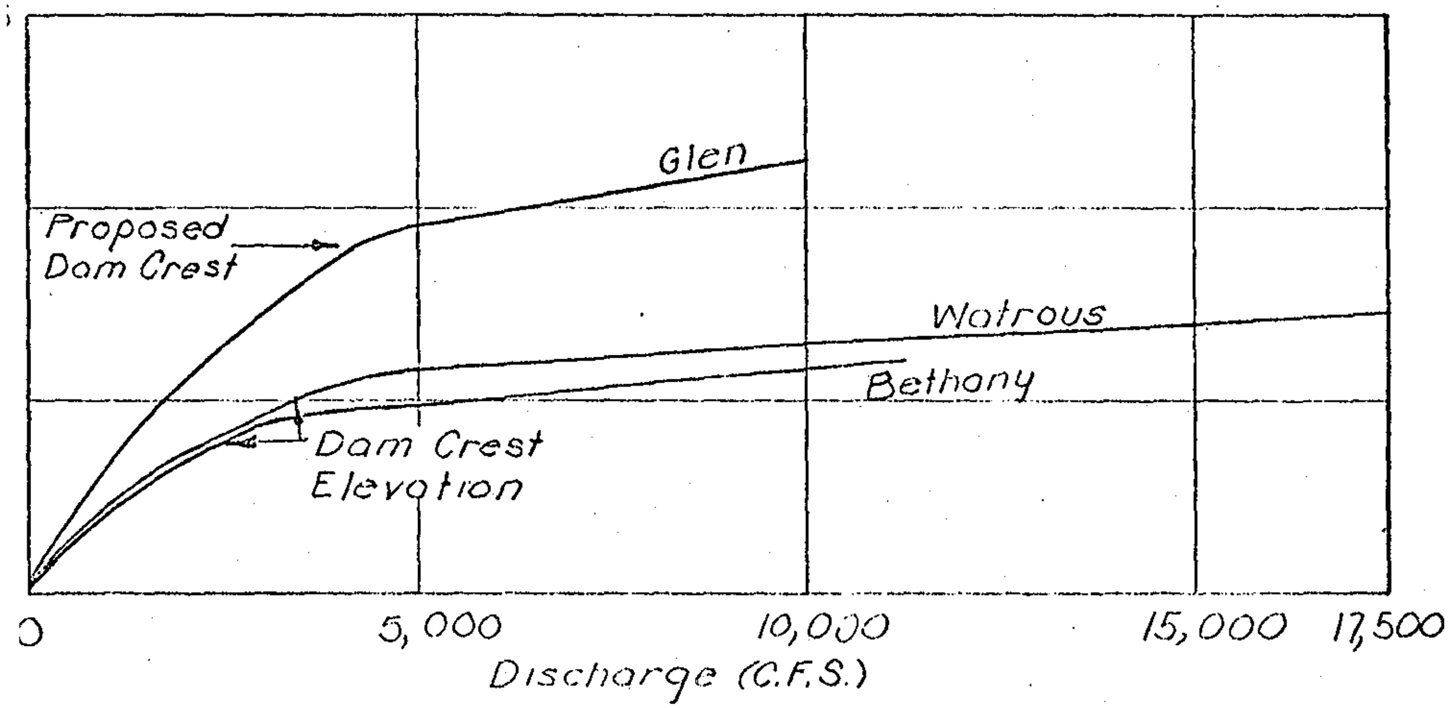
DISTRIBUTION OF 6 AND 12 HR. TOTALS

TIME FROM BEGINNING OF RAIN HOURS	**INCREMENTAL RAINFALL INCHES	** REARRANGED	LESS 1" INITIAL & 0.1" INFILTRATION PER HOUR
1	12.1	0.1	--
2	3.6	0.3	--
3	2.6	1.0	0.3
4	2.2	1.9	1.8
5	1.9	2.6	2.5
6	1.8	12.1	12.0
7	1.0	3.6	3.5
8	0.5	2.2	2.1
9	0.3	1.8	1.7
10	0.2	0.5	0.4
11	0.1	0.2	0.1
12	0.1	0.1	--
	26.4	26.4	24.4

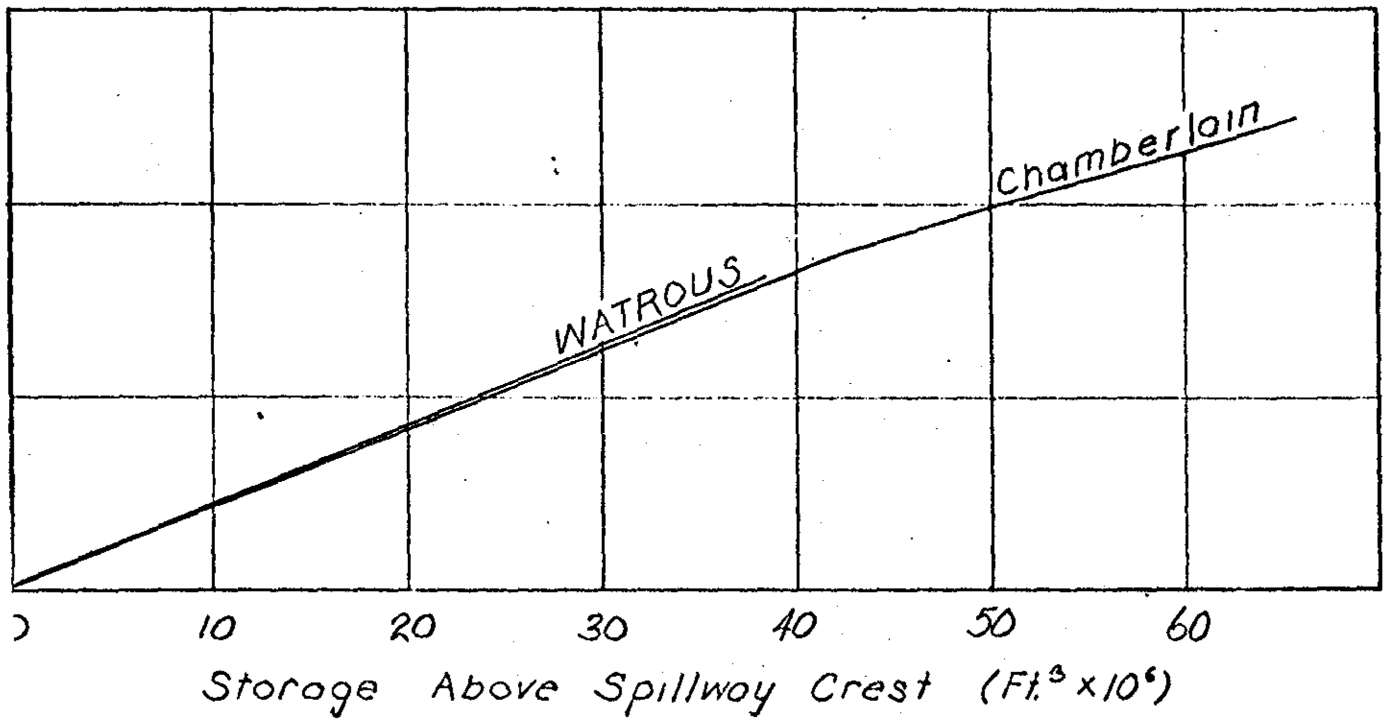
*From Weather Bureau Technical Paper 33 1956

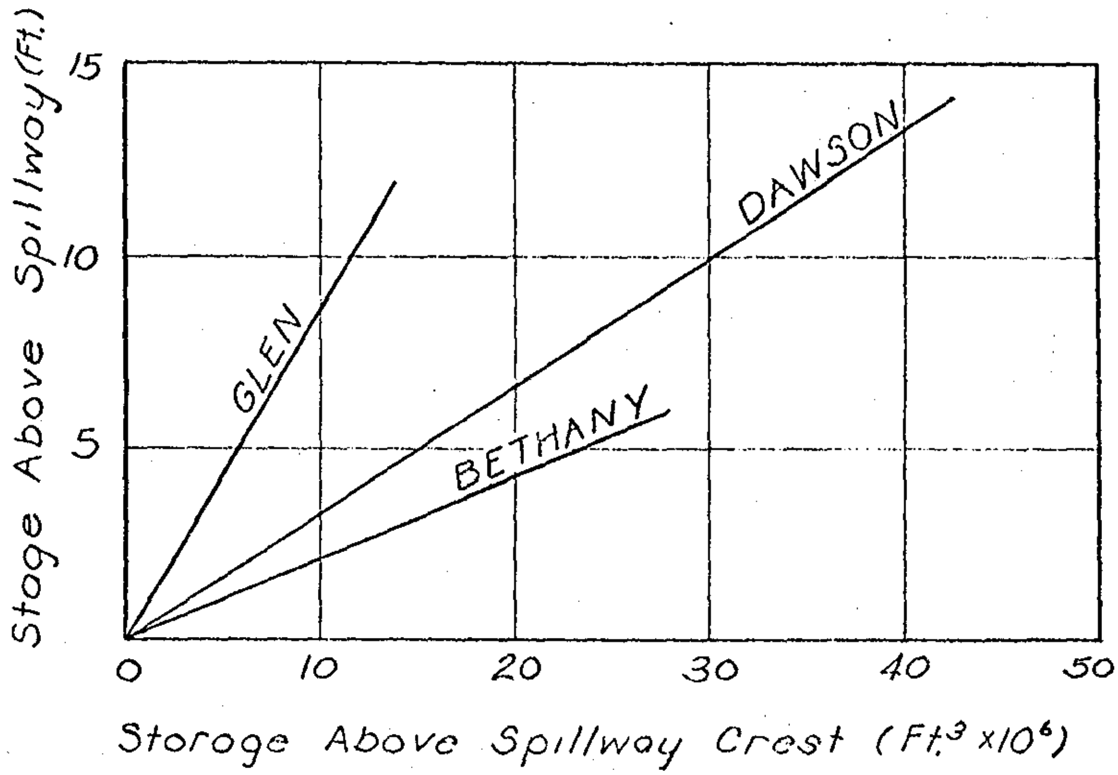
** Distributed and arranged as recommended in U. S. Department of the Interior Publication "Design of Small Dams"





SPILLWAY RATING CURVES





STAGE - CAPACITY
CURVES

BETHANY - MAX POSSIBLE STORM INFLOW HYDROGRAPH

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	
1" STORM	20	200	415	370	280	190	120	80	60	40	20	10	5	3	1	0						
0.3	6	60	125	111	84	57	36	24	18	12	6	3	2	1	0	0						
1.8		36	360	747	656	504	342	216	144	108	72	36	18	9	5	2	0					
2.5			50	500	1038	925	700	475	300	200	150	100	50	25	13	8	3	0				
12.0				240	2400	4980	4440	3360	2280	1440	960	720	480	240	120	60	36	12	0			
2.5					70	700	1450	1300	980	665	420	280	210	140	70	35	18	11	4	0		
2.1						42	420	870	777	589	400	252	168	126	84	42	21	11	6	2	0	
1.7							34	340	706	629	476	323	204	136	102	68	34	17	8	5	2	0
0.4								8	80	166	143	112	76	48	32	24	16	8	4	2	1	0
0.1									2	20	42	37	28	19	12	8	6	4	2	1	0	0
24.4	6	96	535	1598	4248	7208	7422	6593	5287	3829	2674	1863	1236	744	438	247	134	63	42	10	3	

Σ 1" = 1814   1,814 × 24.4 = 44,262   CHECKS   TOTAL = 44,276

CHAMBERLAIN

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21			
1" STORM	15	80	370	475	440	330	220	160	120	95	70	55	38	22	18	10	5	3	1	0				
0.3	5	24	111	143	132	99	66	48	36	29	21	17	12	7	5	3	2	1	0	0				
1.8		27	144	665	855	791	595	396	288	216	171	126	99	69	40	32	18	9	5	2	0			
2.5			38	200	925	1190	1100	825	550	400	300	238	175	138	95	55	45	25	13	8	3	0		
12.0				180	960	4440	5700	5290	3960	2640	1920	1440	1140	840	660	456	264	216	120	60	36	12		
3.5					53	280	1295	1660	1540	1155	770	560	420	332	245	193	133	77	63	35	18	11	4	
2.1						32	168	777	997	925	694	461	336	252	200	147	115	80	46	38	21	10	6	
1.7							26	136	629	807	748	560	374	272	204	162	119	94	65	37	31	17	8	
0.4								6	32	148	190	176	132	88	64	48	38	28	22	15	9	7	4	
0.1									2	8	37	48	44	33	22	16	12	10	7	6	4	2	2	1
24.4	5	51	293	1188	2925	6832	8950	9136	8034	6328	4851	3626	2732	2031	1535	1112	746	540	341	201	122	59		

Σ 1" = 2527   2527 × 24.4 = 61658   CHECKS TOT: 61640

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226 WESTCHESTER AVENUE
WHITE PLAINS, N.Y. 10604

SUBJECT NEW HAVEN - MAX POSSIBLE INFLOW HYDROGRAPH

BY BM DATE 6/28/67

JOB NO. 144700

SHEET NO. 1 OF 8

MALCOLM PIRNIE ENGINEERS
220 WESTCHESTER AVENUE
WHITE PLAINS, N.Y. 10604

BY BM DATE 6/28/65

SUBJECT NEW HAVEN - MAX POSSIBLE INFLOW HYDROGRAPH

EXHIBIT 4

SHEET NO. 2 OF 8

JOB NO. 1447 00

GLEN - MAX POSSIBLE STORM INFLOW HYDROGRAPH

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1" STM	20	130	269	220	140	80	50	30	20	10	5	3	2	1	0						
0.3	6	54	81	66	42	24	15	9	6	3	2	1	0	0	0						
1.8		36	324	486	396	252	144	90	54	36	18	12	6	0	0						
2.5			50	450	672	550	350	200	125	75	50	25	13	8	5	3	0				
12.0				240	2160	3230	2640	1680	960	600	360	240	120	60	36	24	12	0			
3.5				70	630	942	770	490	280	175	105	70	35	18	10	7	4	0	0		
2.1					42	378	569	461	294	168	105	63	42	21	11	6	4	2	0		
1.7						34	306	458	374	238	136	85	51	34	17	8	5	3	2	0	
0.4							8	72	107	88	56	32	20	12	8	4	2	1	1	0	0
0.1								2	18	27	22	14	8	5	3	2	1	6	0	0	0
24.4	6	90	455	1312	2942	5410	4802	3462	2218	1410	854	542	29,5	158	90	54	28	6	3		

Σ1" = 1030   1030 × 24.4 = 25132   CHECKS TOT. = 25137

WATROUS

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	
1" STM	20	220	440	420	320	220	150	110	80	60	40	25	15	10	5	1	0					
0.3	6	66	132	126	96	66	45	33	24	18	12	8	5	3	2	0	0					
1.8		36	396	792	756	576	396	270	198	144	108	72	48	30	18	12	0	0				
2.5			50	550	1100	1050	800	550	375	275	200	150	100	63	38	25	13	3	0			
12.0				240	2640	5280	5040	3840	2640	1800	1320	960	720	480	300	180	120	60	12			
3.5					70	770	1540	1470	1120	770	525	385	280	210	140	88	53	35	18	4		
2.1						42	462	924	881	671	462	315	231	168	126	84	53	32	21	10	2	
1.7							34	374	744	714	544	374	255	187	136	102	68	43	25	17	8	2
0.4								8	88	176	168	124	88	60	44	32	24	16	10	6	4	2
0.1									2	22	44	42	32	22	15	11	8	6	4	3	2	1
24.4	6	102	578	1708	4662	7784	8317	7469	6076	4596	3383	2434	1759	1223	819	534	286	195	90	40	16	5

Σ1" = 2136   2136 × 24.4 = 52118   CHECKS TOTAL = 52076

MALCOLM PIRNIE ENGINEERS
220 WESTCHESTER AVENUE
WHITE PLAINS, N.Y. 10604

BY BM DATE 6/28/67

SUBJECT NEW HAVEN - MAX. POSSIBLE INFLOW HYDROGRAPH

EXHIBIT 4
SHEET NO. 3 OF 8
JOB NO. 14700

DAWSON - MAXIMUM POSSIBLE INFLOW HYDROGRAPH

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1" STM	16	135	170	90	50	30	20	2	1	0											
0.3	3	41	51	27	15	9	6	1	0	0											
1.8		18	246	306	162	90	54	36	6	0											
2.5			25	338	425	225	125	75	50	5	3										
12.0				120	1640	2040	1080	600	360	240	40	12	0								
3.5					35	472	595	315	175	105	70	7	3								
2.1						21	284	357	189	105	63	42	4	2	0						
1.7							17	230	289	153	85	51	34	3	2	0					
0.4							4	54	68	36	20	12	8	1	0	0	0				
0.1							1	14	17	9	5	3	2	0	0	0	0	0			
24.4	3	59	322	791	2277	2857	2166	1682	1154	653	286	127	48	6	2						

24.4 × 508 = 12,400 CHECKS TOTAL = 12,440

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BY B.M. DATE 7/14/67

MALCOLM PIRNIE ENGINEERS
220 WESTCHESTER AVENUE
WHITE PLAINS, N. Y. 10604

EXHIBIT 4

SHEET NO. 4 OF 8

CHKD. BY .... DATE ....

JOB NO. 144700

SUBJECT ROUTING MAX POSSIBLE STORM DISTRIBUTED ACCORDING TO "SMALL DAMS" THROUGH BETHANY

BETHANY RES. & SPILLWAY

TIME	$I_{TOT.}$	$\frac{2S_1}{t} - O$	$\frac{2S_2}{t} + O_2$	O_2	STAGE
0	0	0	6	-	
1.0	6	4	106	1	-
2.0	96	86	717	10	-
3.0	535	667	2800	25	0.1
4.0	1598	2360	8206	220	0.7
5.0	4248	28,080	37,328	1,210	2.5
5.25	4988	29628	40,344	1,350	2.8
5.50	5728	36244	48,440	2,050	3.6
5.75	6468	41340	55,016	3,550	4.3
6.00	7208	44216	58,685	5,400	4.8
6.25	7261	45485	60,060	6,600	5.1
6.50	7314	45660	60,341	7,200	5.2
6.75	7367	45741	60,530	7,300	5.2
7.00	7422	45830	60,466	7,350	5.2
7.25	7214	45766	59,987	7,350	5.2
7.50	7007	45787	59,594	7,100	5.2
7.75	6800	45694	59,087	6,950	5.1
8.0	6593	6900	18,780	6,800	5.1
9.0	5287	7380	16,496	5,700	4.9
10.0	3829	7896	14,399	4,300	4.5
11.0	2674	8099	12,636	3,150	4.2
12.0	1863	7836	10,935	2,400	3.9
13.0	1236	6995	8,975	1,970	3.5
14.0	744			1,400	2.8

MALCOLM PIRNIE ENGINEERS
275 WESTCHESTER AVENUE
WHITE PLAINS, N. Y. 10604

EXHIBIT 4

BM DATE 6/29/67 SHEET NO. 3 OF 8

BY DATE JOB NO. 144700

SUBJECT ROUTING MAX. POSSIBLE STORM DISTRIBUTED ACCORDING TO "SMALL DAMS" THROUGH CHAMBERLAIN RES. & SPILLWAY

CHAMBERLAIN RES. & SPILLWAY

TIME	I_{TOT}	$2S_1/t - O$	$2S_2/t + O$	O	STAGE ON SPILLWAY
0	0	0	5	-	-
1	5	3	59	1	-
2	51	39	383	10	-
3	293	343	1824	20	0.2
4	1,188	1524	5637	150	0.8
5	2925	4637	14394	500	2.0
6	6832	10,394	26,176	2000	4.8
7	8950	16,376	34,464	4900	8.4
8	9138	21,464	38,636	6500	10.1
9	8034	24,216	38,578	7210	10.8
10	6328	24,178	35,357	7200	10.8
11	4851	22,017	30,494	6670	10.2
12	3626	18,894	25,252	5800	9.3
13	2732	15852	20,615	4700	8.2
14	2031	13,815	17,381	3400	6.7
15	1535	12,181	14,828	2600	5.7
16	1112	10,728	12,586	2050	4.9
17	746	9,386	10,672	1600	4.1
18	540	8,232	9,113	1220	3.4
19	341	7.133	7,675	990	3.0
20	201	6095	6418	790	2.7
21	122	4895	5,076	600	2.2'
22	59	4075	4158	410	1.7'
23	24	3458		350	1.5'

EXHIBIT 4

MALCOLM PIRNIE ENGINEERS
226 WESTCHESTER AVENUE
WHITE PLAINS, N. Y. 10604

B.M DATE 7/13/67 SHEET NO. 6 OF 8

BY DATE JOB NO. 144700

SUBJECT ROUTING MAX POSSIBLE STORM DISTRIBUTED ACCORDING TO "SMALL DAMS" THRO' GLEN RES. & SPILLWAY

GLEN RES. & SPILLWAY

TIME	I GLEN	I CHAMB	I TOTAL	$\frac{2S}{t} - O$	$\frac{2S}{t} + O$	O_{TOT}	$O_{B.O.}$	$O_T - O_{B.O}$	STAGE ON SPILLWAY
0	0	0	0	0	7				
1	6	1	7	5	112	1	-	1	
2	90	10	100	72	647	20	-	20	
3	455	20	475	297	2234	175	-	175	0.8
4	1312	150	1462	1054	6958	590	-	590	2.2
5	3942	500	4442	1758	13610	2600	-	2600	6.5
6	5410	2000	7410	-590	16522	7100	285	6815	10.1
7	4802	4900	9702	-2478	17186	9500	"	9115	11.0
8	3462	6500	9962	-2514	16476	9950	"	9665	11.3
9	2218	7210	9428	-2324	15714	9500	"	9215	11.0
10	1410	7200	8610	-1746	14388	8730	"	8445	10.8
1	854	6670	7524	-972	12894	7680	"	7395	10.4
2	542	5800	6342	-106	11231	6500	"	6215	10.0
13	295	4700	4995	811	9364	5210	"	4925	9.5
4	158	3400	3558	1364	7612	4000	"	3715	8.3
5	90	2600	2690	1412	6206	3100	"	2815	6.9
6	54	2050	2104	1406	5138	2400	"	2015	5.5
7	28	1600	1628	1338	4192	1900	"	1615	4.8
8	6	1220	1226	1192	3411	1500	"	1215	3.9
9	3	990	993	1011	2794	1200	"	915	3.0
0	-	790	790	814	2204	990	"	705	2.5
1	-	600	600	604	1614	800	"	515	2.0
2	-	410	410	414	1174	600	"	315	
3	-	350	350			420	"	135	

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JECT ROUTING MAX POSSIBLE STORM ACCORDING TO "SMALL DAMS" THROUGH WATROUS

WATROUS RES & SPILLWAY

TIME	I_{BETH}	$I_{WATROUS}$	I_{TOT}	$\frac{2S_1}{t} - O$	$\frac{2S_2}{t} + O$	O_2	STAGE
0	0	0	0	0	7		
1	6	1	7	5	124	1	-
2	102	10	112	44	759	40	-
3	578	25	603	559	3090	100	-
4	1708	220	1928	2290	10 090	400	1.0
5	4662	1210	5872	7090	25446	1500	3.1
6	5400	7784	13184	56,000	83701	8500	6.1
6.25	6600	7917	14,517	57,701	87468	13000	6.8
6.50	7200	8050	15,250	59,068	89801	14200	6.9
6.75	7300	8183	15483	59,600	90,150	15100	7.0
7.0	7350	8317	15667	59,750	90,872	15200	7.0
7.25	7350	8105	15455	60,072	90,520	15400	7.1
7.50	7100	7893	14,993	59,920	89,544	15300	7.1
7.75	6950	7681	14,631	59,544	88.444	15,000	7.0
8.0	6800	7469	14,269	3,300	29,345	14,700	7.0
9.0	5700	6076	11,776	4,945	25,611	12,200	6.7
10.0	4300	4590	8890	7,211	22,634	9,200	6.3
11.0	3150	3383	6533	9,234	20,601	6,700	6.1
12.0	2400	2434	4834	9,601	18,164	5,500	5.6
13.0	1970	1759	3729	9964	16,316	4,100	5.2
14.0	1400	1223	2623			3,200	4.8

EXHIBIT 4

MALCOLM PIRNIE ENGINEERS
226 WESTCHESTER AVENUE
WHITE PLAINS, N. Y. 10604

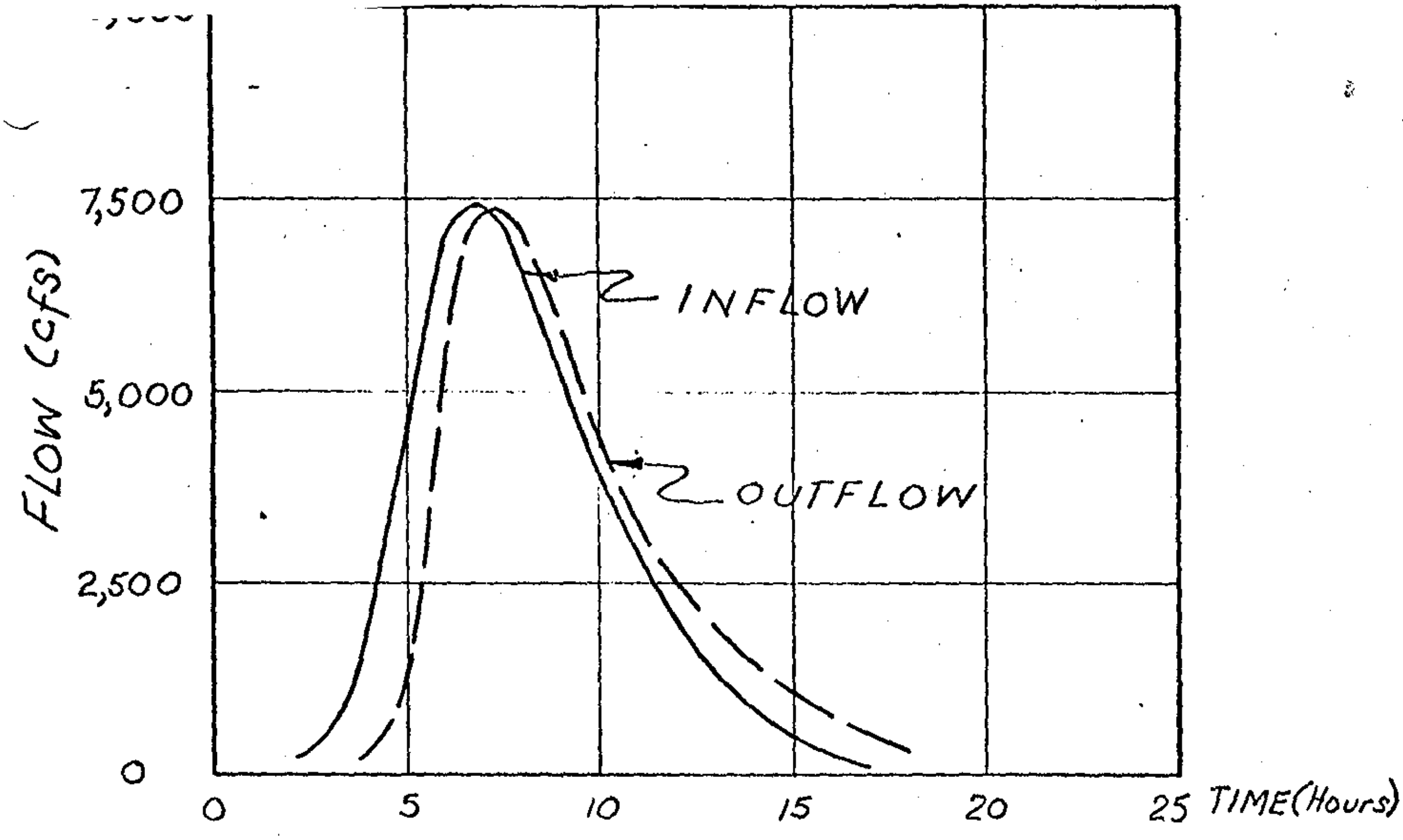
B.M. DATE 7/14/67 SHEET NO. 8 OF 8

CK. BY DATE JOB NO. 144700

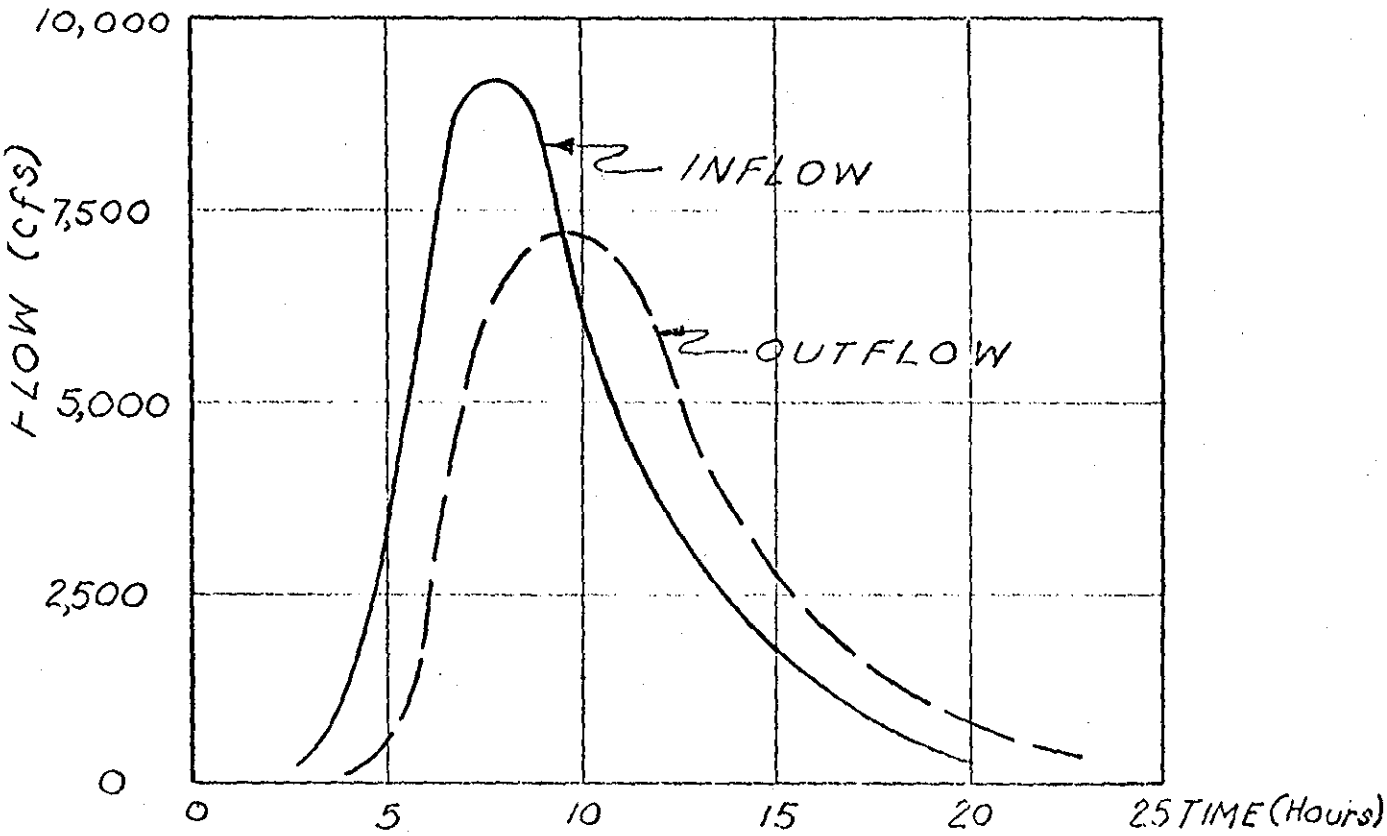
SUBJECT ROUTING MAX. POSSIBLE STORM, DISTRIBUTED ACCORDING TO "SMALL DAMS" THROUGH DAWSON

DAWSON RES & SPILLWAY

TIME	I_{TOT}	$\frac{2S_1}{t} - O_1$	$\frac{2S_2}{t} + O_2$	O_{TOT}	O_{BO}	$O_{TOT} - O_{BO}$	STAGE
0		0	1				
0.5	1	1	7	0	–	0	–
1.0	5	5	39	1	–	1	–
1.5	29	35	183	2	–	2	–
2.0	119	163	502	10	–	10	–
2.5	220	402	1219	50	–	50	0.6
3.0	597	919	2488	150	–	150	0.9
3.5	972	1888	4641	300	–	300	1.3
4.0	1781	3841	9256	400	–	400	1.4
4.5	3634	7256	17267	1000	–	1000	2.6
5.0	6377	12,667	30,457	2300	–	2300	4.6
5.5	11,413	19,457	49,322	5,500	–	5,500	7.4
6.0	18,457	62,600	104,442	11,700	740	10,960	11.2
6.25	23,385	65,842	114,239	19,300	"	18,560	12.8
6.50	25,012	65,239	116,590	24,500	"	23,760	13.5
6.75	26,339	65,190	118,395	25,700	"	24,960	13.7
7.0	26,866	64,795	119,718	26,300	"	25,560	13.8
7.25	27,057	65,718	119,723	27,000	"	26,260	13.9
7.50	26,948	65,723	119,310	27,000	"	26,260	13.9
7.75	26,639	65,910	118,831	26,700	"	25,960	13.8
8.0	26,332	65,681	117,468	26,600	"	25,860	13.8
8.25	25,455	64,868	114,911	26,300	"	25,560	13.8
8.50	24,588	65,311	113,620	24,800	"	24,060	13.6
8.75	23,721	64,420	112,045	24,100	"	23,360	13.5
9.0	22,854	21,400	64,979	23,500	"	22,760	13.4
9.5	20,725	22,979	62,237	21,000	"	20,260	13.0
10.0	18,583	23,887	59,094	19,200	"	18,460	12.8
10.5	16,624	24,494	55,784	17,300	"	16,560	12.4
11.0	14,666	25,384	53,240	15,200	"	14,460	12.0
11.5	13,190	25,240	51,430	14,000	"	13,260	11.9
12.0	13,000			12,800	"	12,060	11.6
12.5							

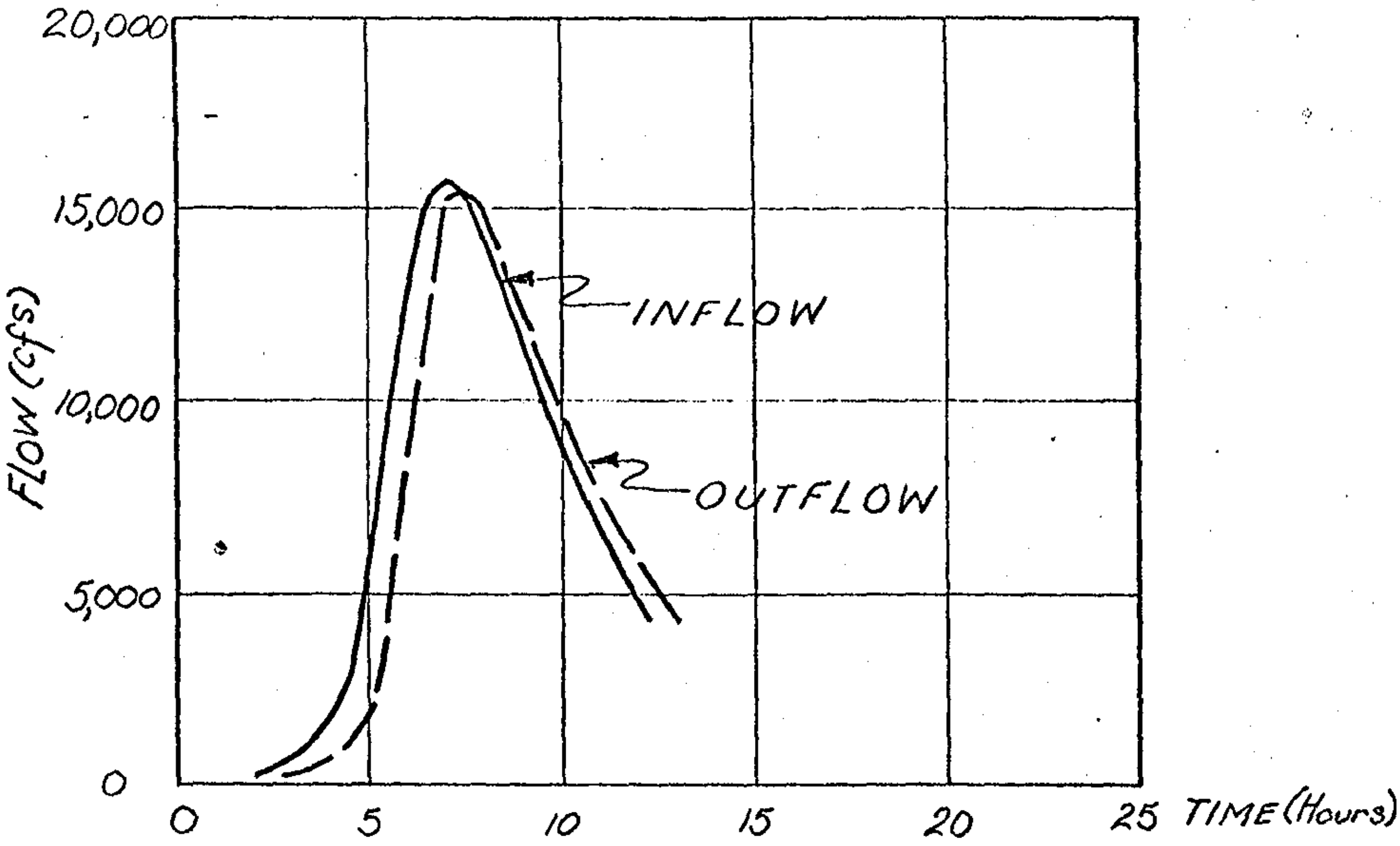


BETHANY

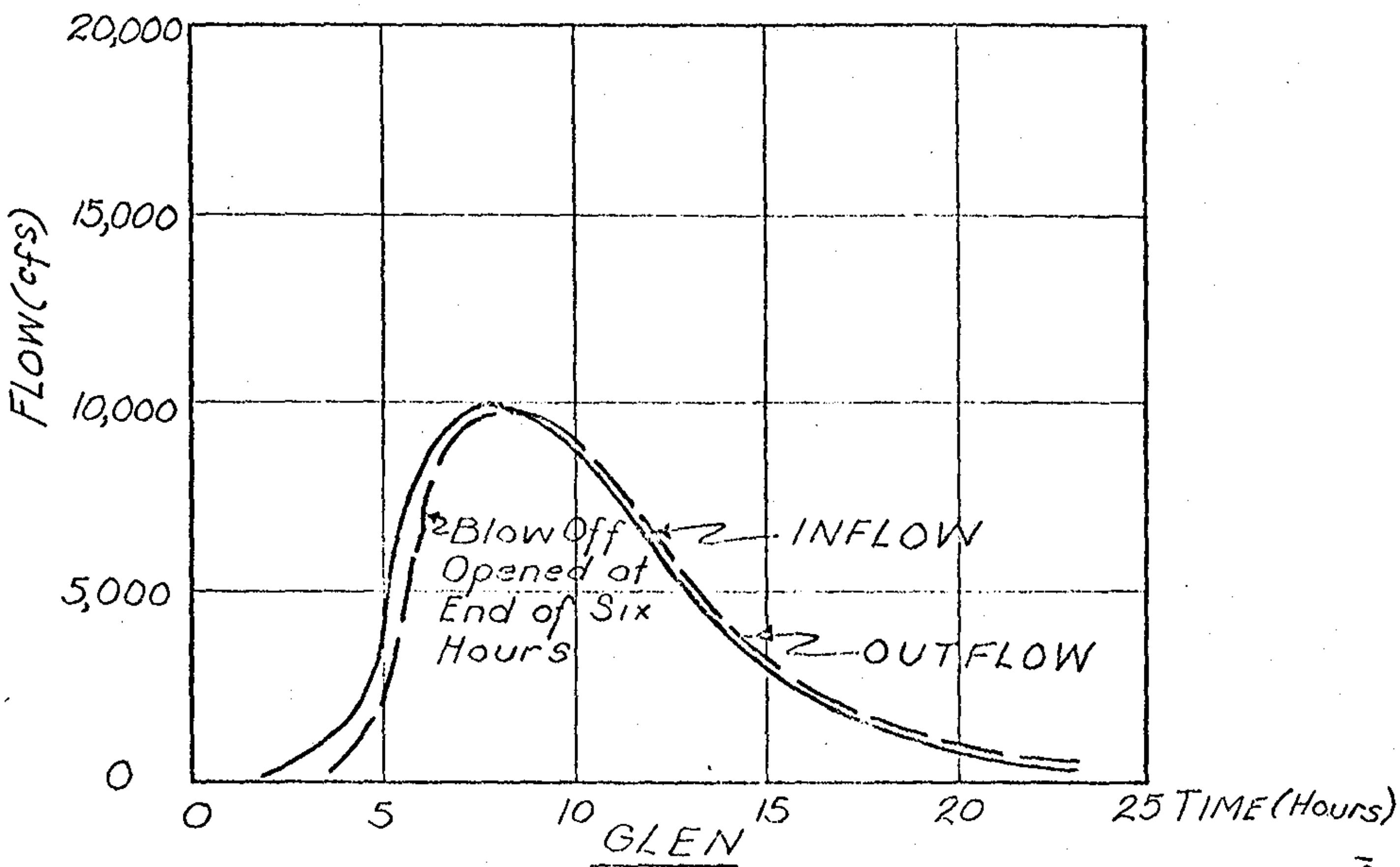


CHAMBERLAIN

INFLOW-OUTFLOW CURVES

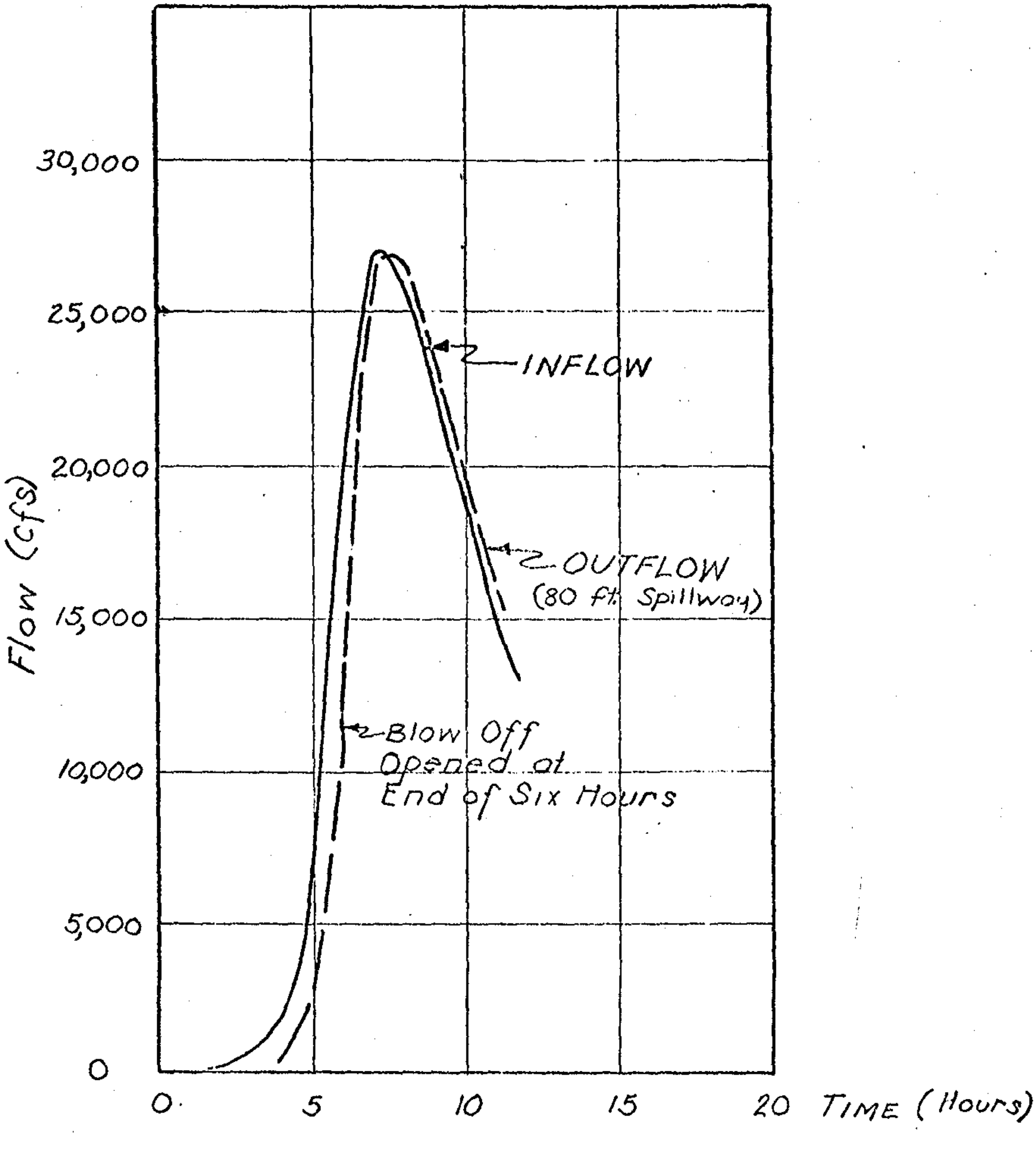


WATROUS



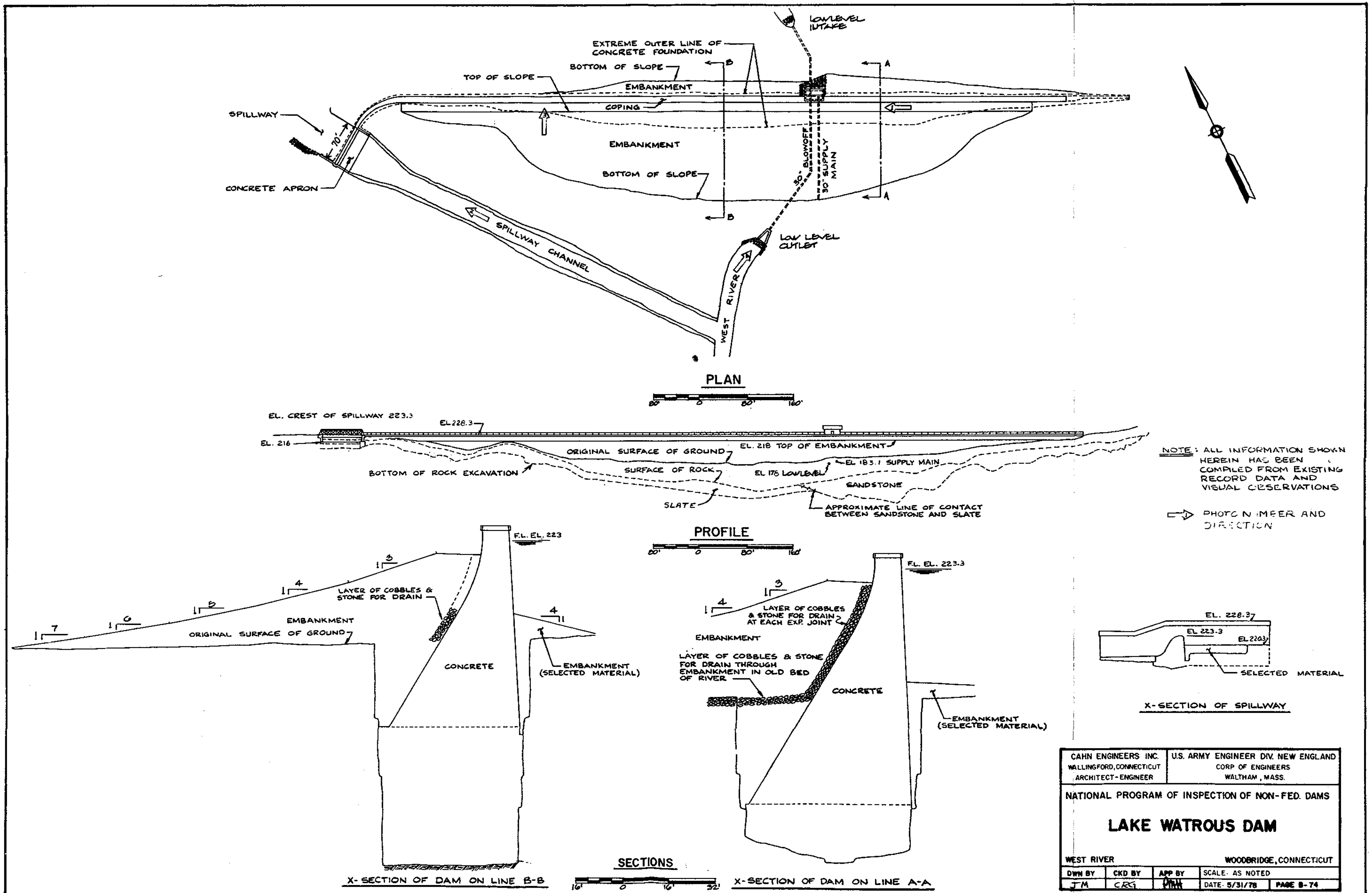
GLEN

INFLOW - OUTFLOW



~~BETHANY~~
DAWSON

INFLOW - OUTFLOW
CURVES



CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ARCHITECT-ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORP OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LAKE WATROUS DAM

WEST RIVER — WOODBRIDGE, CONNECTICUT

DWN BY	CKD BY	APP BY	SCALE: AS NOTED
JM	CRG	[illegible]	DATE: 5/31/78 — PAGE B-74

APPENDIX

SECTION C: DETAIL PHOTOGRAPHS

PHOTO NO.1 - General view of downstream face of dam and embankment.

PHOTO NO.2 - Spillway, bridge, and channel cut into natural rock formation.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	LAKE WATROUS DAM WEST RIVER WOODBRIDGE, CONNECTICUT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER		CE# 27 531 GD DATE 5/31/78 PAGE C-1

PHOTO NO.3 - Seepage and staining of downstream face of dam at horizontal construction joint.

PHOTO NO. 4 - Low level outlet structure. Note fallen tree.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	LAKE WATROUS DAM WEST RIVER WOODBRIDGE, CONNECTICUT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER		CE # 27 531 GD DATE 5/31/78 PAGE C-2

APPENDIX

SECTION D: HYDRAULIC/HYDROLOGIC COMPUTATIONS

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

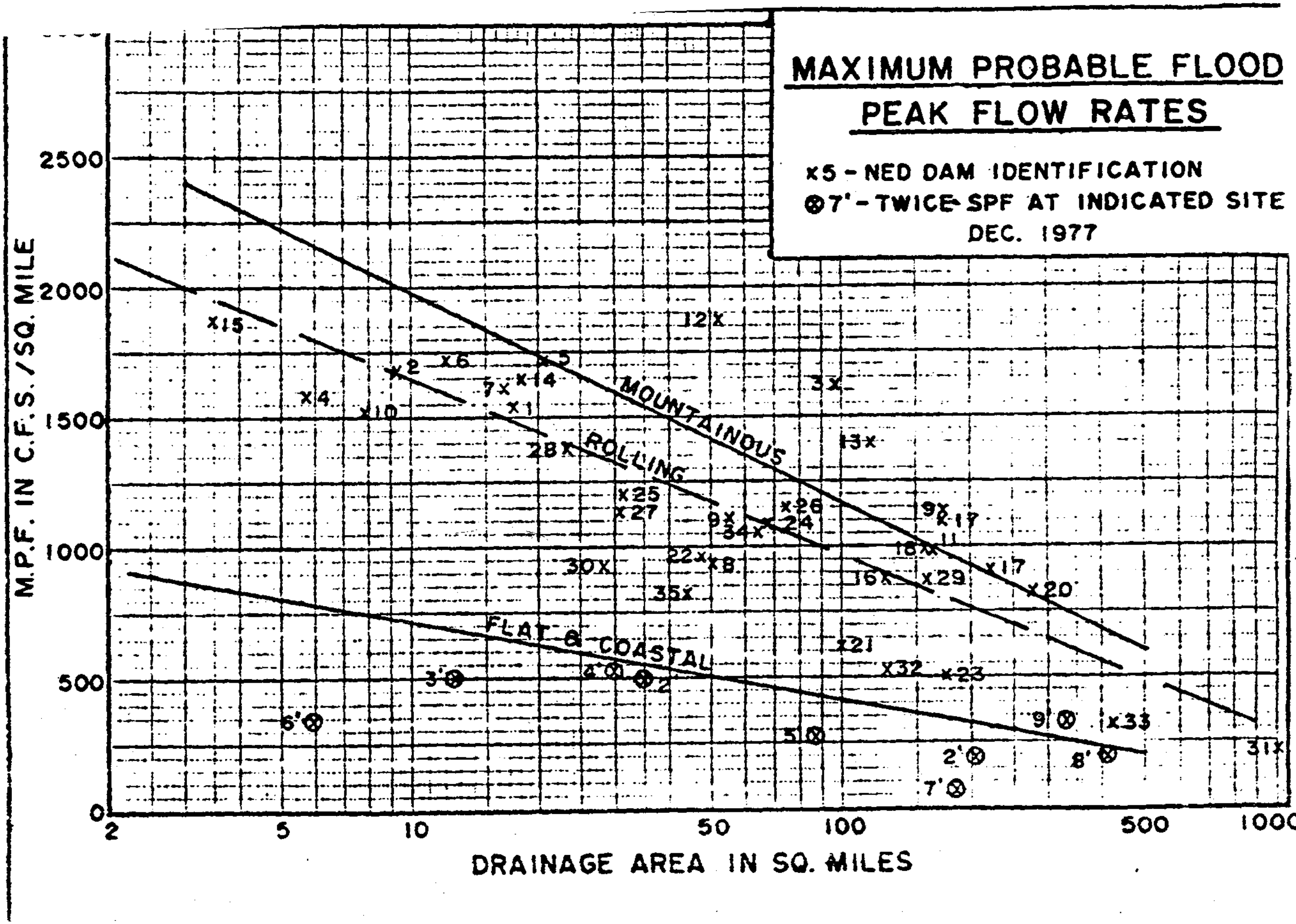
March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

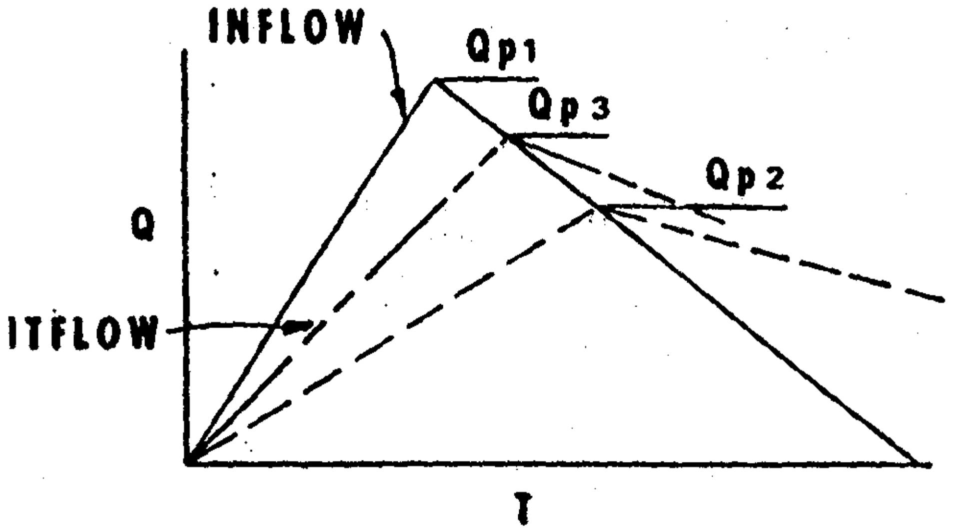
MAXIMUM PROBABLE FLOWS BASED ON TWICE THE STANDARD PROJECT FLOOD (Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330



MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITE
DEC. 1977
M.P.F. IN C.F.S. / SQ. MILE
DRAINAGE AREA IN SQ. MILES
MOUNTAINOUS
ROLLING
FLAT & COASTAL
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guid Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

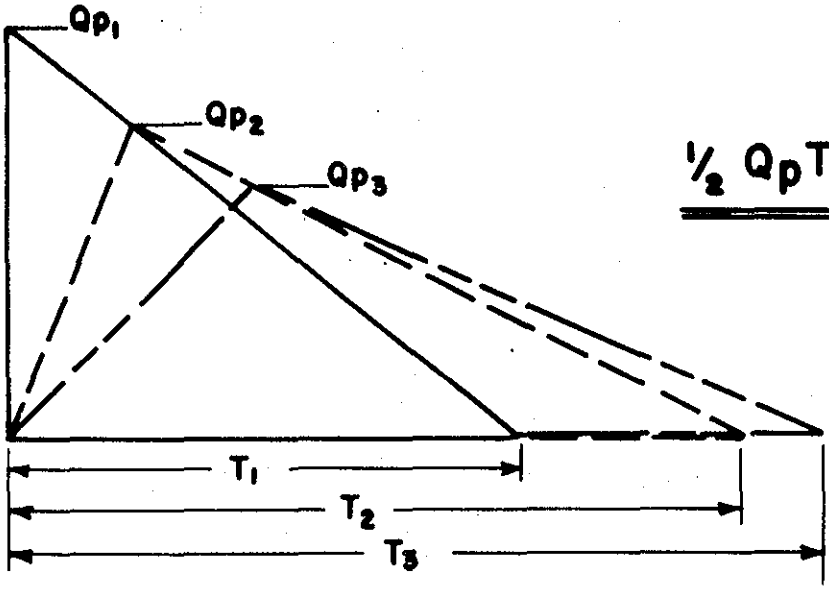
c. Maximum Probable Flood Runoff In Ne England equals Approx. 19", Therefor

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".

"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\frac{1}{2} Q_p T = 12\,S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g}\, Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1\left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1\left(1 - \frac{V_{ave}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

HYDROLOGIC / HYDRAULIC INSPECTION
LAKE WATROUS WOODBRIDGE, CONNECTICUT

(1) MAXIMUM PROBABLE FLOOD – PEAK FLOOD RATE

(a) WATERSHED CLASSIFIED AS "ROLLING TO MOUNTAINOUS" TYPE. USE MPF ROLLING TYPE CURVE FURNISHED BY THE ACE, NEW ENGLAND DIV OFFICE FOR THE DETERMINATION OF PMF.

(b) WATERSHED AREA: D.A. = 6.6 SQ. MI (NEW HAVEN WATER CO DATA AUG 1974)
(C.E. MEASURED: 6.9 SQ. MI)
USE D.A. = 7.0 SQ MI (J.W. CONE INSPECTION REPORT 6/26/75 D.A. = 7.0 SQ. MI)

(c) FROM GUIDE CURVE:
M.P.F ≅ 1800 CFS / SQ. MI

(d) M.P.F = PEAK INFLOW
Q = 1800 × 7.0 = 12,600 CFS

(2) SPILLWAY DESIGN FLOOD (SDF)

(a) CLASSIFICATION OF DAM ACCORDING TO ACE RECOMMENDED GUIDELINES.

(i) SIZE* (IMPOUNDMENT): STORAGE (MAX) = 2,780 AC-FT (INTERM.)
HEIGHT = 51 FT (INTERM.)

* FROM NEW HAVEN WATER CO. DATA AUG. 1974, AND TABULATIONS BY ENGR. ALBERT B. HILL (1917/1923) – (US INVENTORY OF DAMS MAX. STORAGE = 4250 AC-FT)
STORAGE TO SPILLWAY = 725.5 MG ≅ 2230 AC-FT.
AREA AT FLOWLINE = 109.1 AC. ∴ ADD. STOR. TO TOP OF DAM = 109.1 × 5 = 550 AC-FT.
MAX. STORAGE ≅ 2780 AC FT. THEREFORE, THE DAM IS CLASSIFIED AS OF "INTERMEDIATE" SIZE.

D-1

HYDROLOGIC/HYDRAULIC INSPECTION

LAKE WATROUS WOODBRIDGE, CONN

(2) (cont'd) - SPILLWAY DESIGN FLOOD (SDF)

(a) CLASSIFICATION OF DAM

(i) HAZARD POTENTIAL:

THE DAM IS LOCATED UPSTREAM OF LAKE DAWSON DAM, WILBUR CROSS PKWY, AND WOODBRIDGE URBAN DEVELOPED AREA. HENCE, THE HAZARD POTENTIAL IS "HIGH"

(ii) SDF

ACCORDING TO ACE RECOMMENDED GUIDELINES FOR LAKE WATROUS THE SDF SHALL BE THE MPF

SDF = MPF = 12,600 CFS

(3) EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES

(a) PEAK INFLOW (SDF = MPF)

Q_{P1} = 12,600 CFS

(b) SURCHARGE HEIGHT TO PASS Q_{P1}

(i) ESTIMATE SURCHARGE ABOVE SPILLWAY CREST

SPILLWAY DATA: (FROM "AS-BUILT" PLANS - LAKE WATROUS DAM, NEW HAVEN WATER CO., JAN. 1915)

LENGTH OF SPILLWAY CREST = 70'

U/S BATTER SLOPE (V:H) = 1.0 : 0.15, ROUNDED OGEE TYPE SPILLWAY. D/S FACE SLOPE (V:H) = 1.0 : 0.6. U/S HEIGHT OF SPILLWAY CREST TO GROUND P = ±3'

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS DAM WOODBRIDGE, CONN

(3)(Cont'd) EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES.

(b) SURCHARGE HEIGHT TO PASS Q_{p1}

(i) ESTIMATE OF SURCHARGE ABOVE SPILLWAY CREST FOR THE EXPECTED HIGH HEAD OVER THE SPILLWAY,

ASSUME $C \simeq 3.6$ $Q \simeq 250\ H^{3/2}$

$$H \simeq \left(\frac{Q}{250}\right)^{2/3}$$

$\therefore$ @ $Q_{p1} = 12{,}600$ CFS

$H_1 \simeq 13.6'$

MAXIMUM FREEBOARD FROM SPILLWAY CREST (ELEV.* 223.3' MSL) TO THE TOP OF THE DAM (ELEV.* 228.3' MSL) IS 5 ft.

HENCE, THE DAM IS OVERTOPPED.

– SPILLWAY CAPACITY AT $H = 5'$, $Q \simeq 2800$ CFS

(ii) COMPUTE TRUE SURCHARGE HEIGHT H_1

DEPTH OF HEAD WATER ABOVE THE DAM $= H_1 - 5$

TOP WIDTH OF MAIN SECTION $= 10'$

NOTE: NEW HAVEN WATER CO. DATA GIVE ELEVATIONS IN NEW HAVEN DATUM (MEAN HIGH WATER) MSL (U.S.C.G.S DATUM) ≡ NEW HAVEN DATUM (MHW) +3.31'

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS DAM WOODBRIDGE, CONN

13) (cont'd) EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES

(b) SURCHARGE HEIGHT TO PASS Q_{P1}

(i) COMPUTE TRUE SURCHARGE HEIGHT H_1

ASSUME $C \approx 2.7$

LENGTH OF MAIN SECTION $L \approx 1204'$

$CL \approx 3250$

$Q \approx 3250 (H_1 - 5)^{3/2}$

A BERM AT THE EASTERLY END RISES 10' IN A DISTANCE OF 90'

EQUIVALENT LENGTH $= \frac{2}{3}\left(\frac{90}{10}\right)(H_1 - 5)$

ASSUME $C \approx 2.6$

$Q \approx 16 (H_1 - 5)^{5/2}$

A BERM AT THE WESTERLY END RISES 10' IN A DISTANCE OF ±150'

EQUIVALENT LENGTH $= \frac{2}{3}\left(\frac{150}{10}\right)(H_1 - 5)$

ASSUME $C \approx 2.6$

$Q \approx 26 (H_1 - 5)^{5/2}$

THEREFORE, DISCHARGE WITH A SURCHARGE OF H_1 ABOVE THE SPILLWAY IS

$$Q \approx 250 H_1^{3/2} + 3250 (H_1 - 5)^{3/2} + 42 (H_1 - 5)^{5/2}$$

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS DAM WOODBRIDGE, CONN.

(3) (CONT'D) EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES

(b) SURCHARGE HEIGHT TO PASS Q_{p1}

(i) COMPUTE THE TRUE SURCHARGE HEIGHT H_1

$$Q_{p1} = 12{,}600 \text{ CFS}$$

$$H_1 \approx 6.82' \text{ SAY } 6.8'$$

THE HEAD WATER ABOVE THE TOP OF THE DAM IS ±1.8'.

(c) VOLUME OF SURCHARGE

ASSUME NORMAL POOL LEVEL 0.5' ABOVE THE SPILLWAY CREST

AREA OF POOL AT FLOWLINE = 109 AC.
(SEE p. 1)

VOL. OF SURCHARGE: WITH $Q_{p1} = 12{,}600$ CFS
$H_1 \approx 6.8'$

IS $109(6.8 - 0.5) = 687$ AC-FT

$D.A = 7.0$ SQ. MI

$$S_1 = \frac{687}{7.0 \times 53.3} = 1.8''$$

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS DAM WOODBRIDGE, CONN.

(3) (CONT'D) EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES

(d) PEAK OUTFLOW FOR SURCHARGE S_1
(SEE GUIDELINES RECOMMENDED BY ACE NEW ENGLAND DIV.)

$$Qp_2 = Qp_1 \left(1 - \frac{S_1}{19}\right)$$

$$\therefore Qp_2 = 12,600 \left(1 - \frac{1.8}{19}\right)$$

$$Qp_2 \approx 11400 \text{ CFS}$$

FOR $Qp_2 = 11,400$ CFS
$H_2 \approx 6.66' \approx 6.7'$
AND $S_2 \approx 1.8''$ $S_{AVE} = 1.8''$

(e) RESULTING PEAK OUTFLOW
$\therefore Qp_3 \approx 11,400$ CFS

AND
$H_3 \approx 6.7'$

(f) SUMMARY:
PEAK INFLOW $Qp_1 = MPF = 12,600$ CFS
PEAK OUTFLOW $Qp_3 = 11,400$ CFS
AVERAGE SURCHARGE ABOVE THE SPILLWAY CREST IS $\pm 6.7'$, IT IS $\pm 1.7'$ ABOVE TOP OF THE DAM. (ELEV. 228.3' MSL) OR W.L. EL. 230' MSL

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATNOUS WOODBRIDGE, CONN

DOWNSTREAM FAILURE HAZARD

(1) ESTIMATE OF D/S DAM FAILURE HYDROGRAPH
(SEE ACE "RULE OF THUMB" GUIDELINES FOR ESTIMATING THE HYDROGRAPHS)

(a) ESTIMATE OF RESERVOIR STORAGE AT TIME OF FAILURE
(SEE D. SHEN COMPS. 5/18/78)

(i) MAXIMUM STORAGE CAPACITY = 2780 Ac-ft

(ii) HEIGHT OF STRUCTURE ABOVE SPILLWAY = 5 ft

(iii) HEIGHT OF MAXIMUM POOL = 51 ft

(iv) ESTIMATED VOLUME OF STORAGE AT TIME OF FAILURE:

TO SURCHARGE* ELEV. ± 230.0' MSL i.e. ± 6.7' ABOVE THE SPILLWAY CREST OR 1.7' ABOVE THE DAM

$S \approx {}^{*1}109.1 \times (6.7) + 2230$

$\approx \underline{\underline{2,960 \text{ Ac-ft}}}$

$\frac{S}{2} = 1480$ Ac-ft

*1: AREA AT FLOWLINE

NOTE: NEW HAVEN WATER Co. DATA GIVE ELEVATIONS IN NEW HAVEN DATUM (MHW). MSL (USCGS DATUM) = NEW HAVEN DATUM (MHW) + 3.31'

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS WOODBRIDGE, CONN

DOWNSTREAM FAILURE HAZARD

(1) (cont'd) - ESTIMATE OF DOWNSTREAM DAM FAILURE HYDROGRAPHS

(b) PEAK FLOOD OUTFLOW Q_{P1}

(i) BREACH WIDTH.

FROM THE NEW HAVEN WATER CO. AS-BUILT PLANS, JAN. 1915 # J-375

TAKING LOWEST ELEV. OF THE ORIGINAL GROUND SURFACE AS DATUM.

TOTAL LENGTH OF DAM AT MID-HEIGHT 760 Ft

$W \approx 0.4(760) \approx 304$ Ft

TAKE $W_b \approx$ 300 Ft

(ii) TOTAL HEIGHT AT TIME OF FAILURE

HEIGHT OF DAM = 51'

SURCHARGE = 1.7'

$Y_0 = 52.7'$

APPROX. WAVE HEIGHT IMMEDIATELY D/S OF DAM SITE

$Y \approx 0.44 Y_0 \approx 23.2'$

(iii) PEAK FLOOD OUTFLOW Q_{P1}

$$Q_{P1} = \frac{8}{27}\sqrt{g}\, W_b\, y_0^{1.5}$$

$= \underline{\underline{193,000 \text{ CFS}}}$

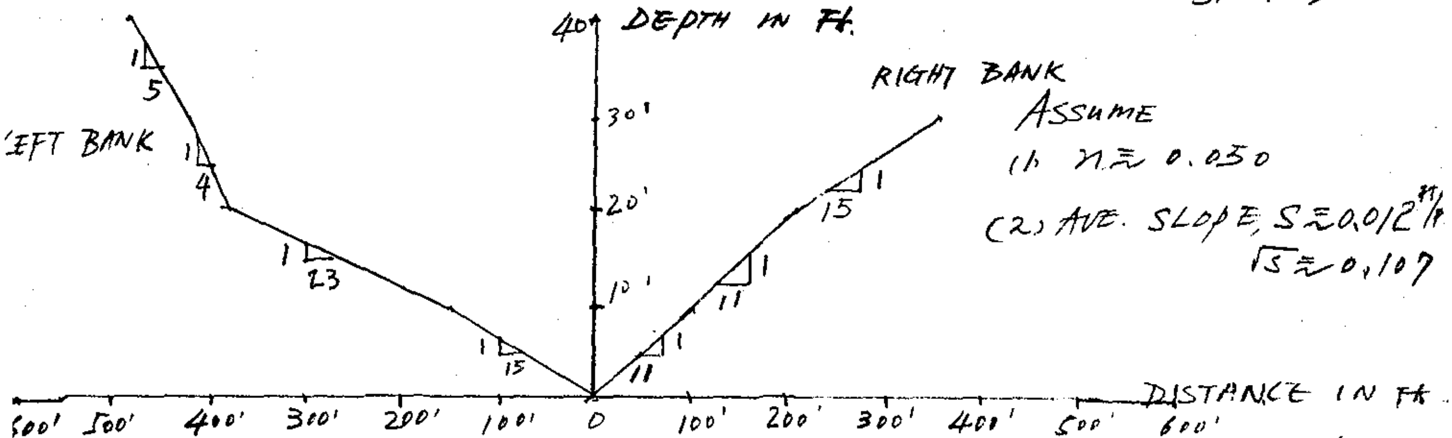
HYDROLOGIC/HYDRAULIC INSPECTION

LAKE WATROUS WOODBRIDGE, CT

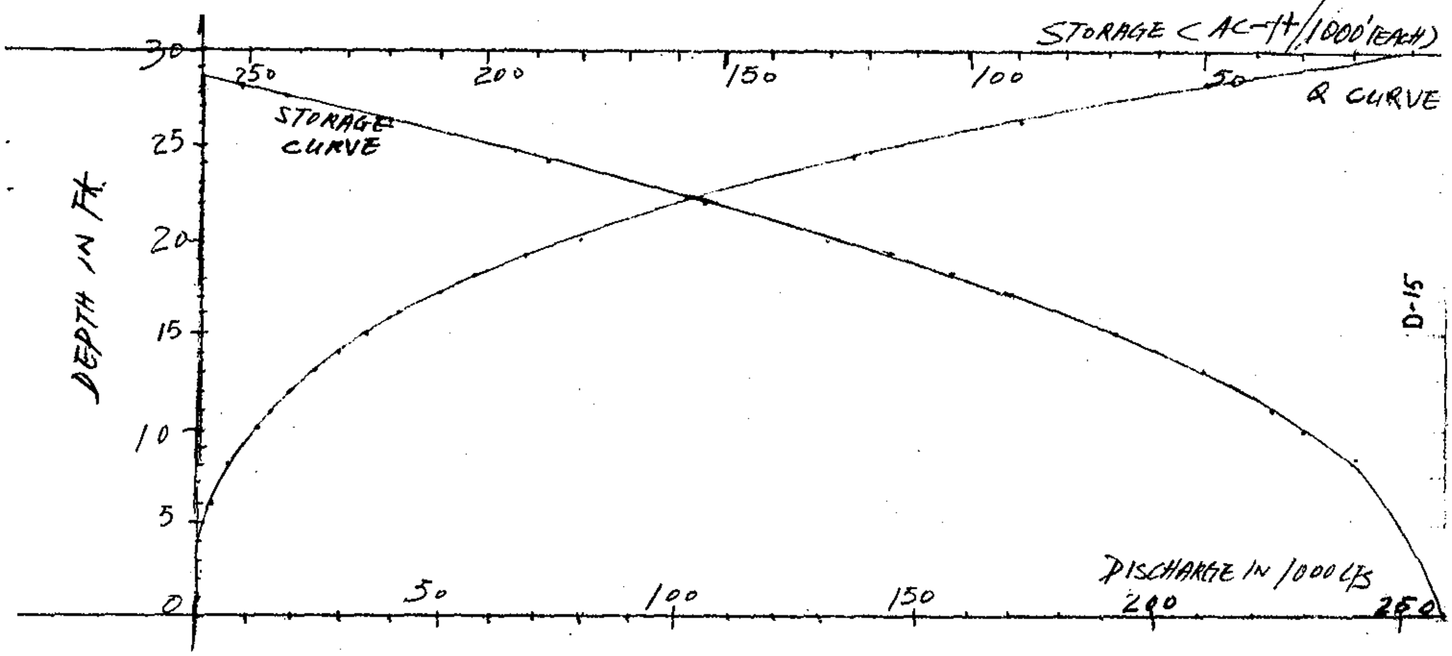
DOWNSTREAM DAM FAILURE HAZARD

(1) (Cont'd) ESTIMATE OF D/S DAM FAILURE HYDROGRAPH

(c) TYPICAL D/S CROSS-SECTION & RATING CURVES
(FROM U.S.G.S WOODBRIDGE AND NEW HAVEN QUADRANGLE SHEETS)



ASSUME
(1) $n \approx 0.050$
(2) AVE. SLOPE, $S \approx 0.012$ ft/ft
$\sqrt{S} \approx 0.107$



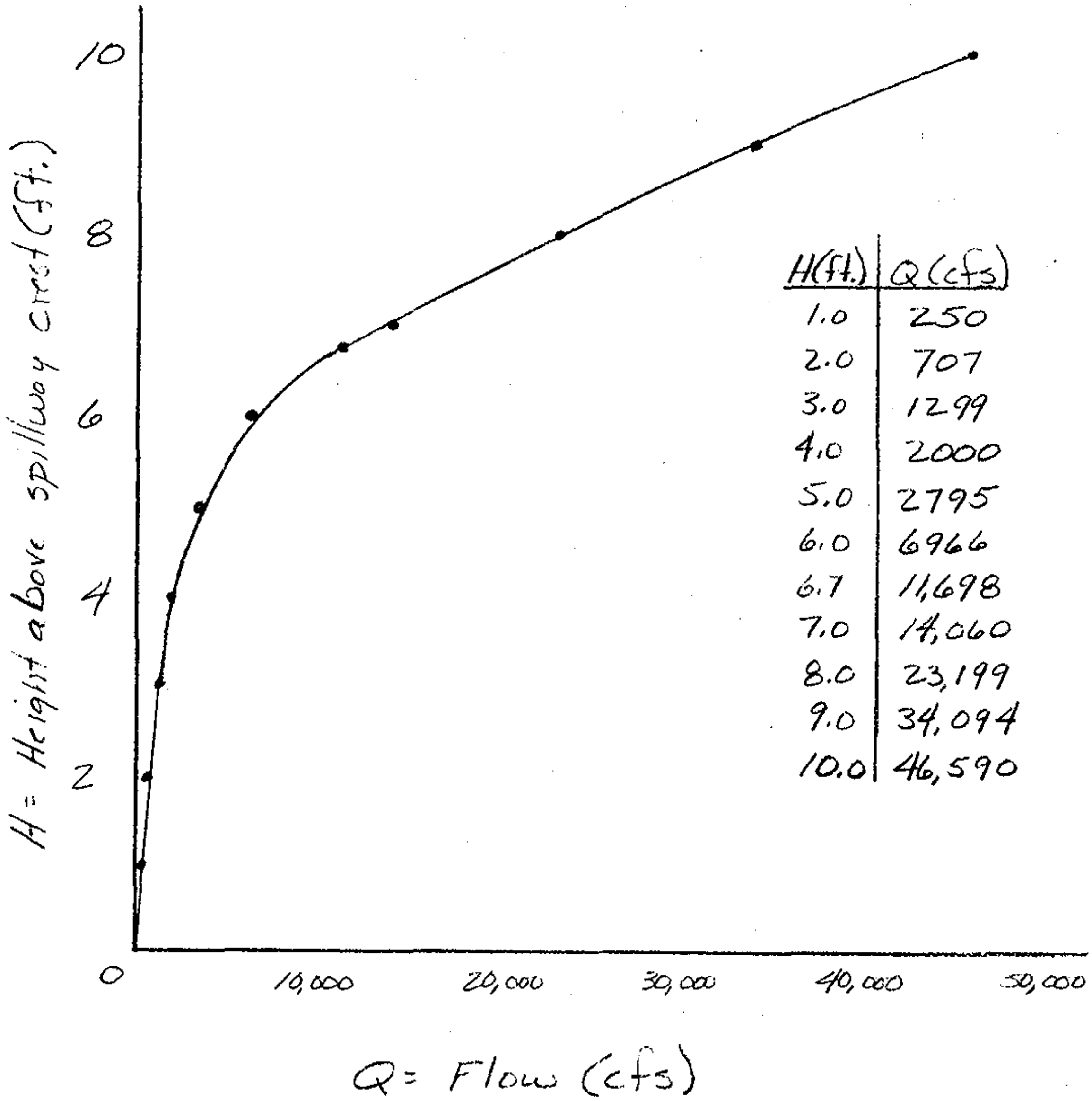
D-15

Cahn Engineers Inc. **Consulting Engineers**

Project: LAKE WATROUS DAM
Sheet 1 of 1
Computed By: HM/CRG
Checked By:
Date: 9/8/78
Field Book Ref.:
Other Refs.:
Revisions:

SPILLWAY RATING CURVE

$$Q = 250H_1^{3/2} + 3250(H_1-5)^{3/2} + 42(H_1-5)^{5/2}$$



H(ft.)	Q(cfs)
1.0	250
2.0	707
3.0	1299
4.0	2000
5.0	2795
6.0	6966
6.7	11,698
7.0	14,060
8.0	23,199
9.0	34,094
10.0	46,590

ahn Engineers Inc. Consulting Engineers

ct INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND Sheet 4 of 7
uted By D. SHEN Checked By HW Date 6/2/1978
Book Ref. Other Refs. CE#27-531-GD Revisions

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS WOODBRIDGE, CT

DOWNSTREAM DAM FAILURE HAZARD

(1) (Cont'd) ESTIMATE OF D/S DAM FAILURE HYDROGRAPH

(d) REACH OUTFLOW (Q_{p2})

(i) @ Q_{p1} = 193,000 CFS. FROM RATING CURVE STAGE ≈ 27.5 FT.

∴ VOLUME IN REACH ≈ 243 × 3.3 (DISTANCE ≈ 3300 FROM LK. WATROUS TO LK DAWSON) FROM TOPO. MAP
≈ 800 Ac-ft

∴ V ≈ 800 Ac-ft < $\frac{S}{2}$ O.K.

(ii) Q_{p2}:

TRIAL $Q_{p2} = Q_{p1}(1-\frac{V}{S}) \approx 193{,}000\ (1-\frac{800}{2960})$

≈ 141,000 CFS

STAGE ≈ 24.5'

V ≈ 3.3 × 193 ≈ 640 Ac-ft

(iii) AVE. VOL. IN REACH ≈ $\frac{640+800}{2}$ ≈ 720 Ac-ft

$Q_{p2} = Q_{p1}(1-\frac{V}{S}) = 193{,}000\ (1-\frac{720}{2960})$

≈ 146,000 CFS

STAGE ≈ 25 ft

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS WOODBRIDGE, CT

DOWNSTREAM DAM FAILURE HAZARD

(1) (CONT'D) ESTIMATE OF D/S DAM FAILURE HYDROGRAPH

(2) ESTIMATE EFFECT OF LAKE DAWSON ON Q_{P_2}

(i) Q_{P2} = INFLOW FLOOD TO RESERVOIR

(SEE J.W. CONE 1965 REPORT CONCERNING DAMS OWNED BY NEW HAVEN WATER CO. ON THE WEST & SARGENT RIVERS)

LENGTH OF SPILLWAY = 80'

MAXIMUM FREEBOARD = 6'

ASSUME $C \cong 3.2$ (CONE: $Q \cong 2870$ CFS @ $H = 5'$)

$$Q = (3.2)(80) H^{3/2} \cong 260 H^{3/2}$$

(ii) SURCHARGE HEIGHT ABOVE SPILLWAY CREST

TOTAL LENGTH OF DAM AND SIDE SPILLS = 1000'

ASSUME $C \cong 2.7$

$$Q \cong (2.7)(1000)(H-6)^{3/2}$$

HENCE, DISCHARGE IS

$$Q \cong 260 H^{3/2} + 2700 (H-6)^{3/2}$$

$\therefore$ @ $Q_p = 146{,}000$ CFS

$H_2 \cong 19'$

HYDROLOGIC/HYDRAULIC INSPECTION

LAKE WATROUS WOODBRIDGE, CT

DOWNSTREAM DAM FAILURE HAZARD

1) (CONT'D) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

2) ESTIMATE EFFECT OF LAKE DAWSON ON Q_{p2}

(iii) EFFECT OF STORAGE OF LAKE DAWSON

AREA OF POOL AT FLOWLINE . 69 Ac (J.W. CONE REPORT 1965)

ASSUME NORMAL POOL 0.5' ABOVE FLOWLINE

∴ VOL. OF SURCHARGE

$V_R = 69 \times (19 - 0.5) \doteqdot 1280$ Ac-ft $< \frac{S}{2}$

(iv) PEAK FLOOD OUTFLOW, TRIAL Q_{p3}

$Q_{p3} = Q_{p2}(1 - \frac{V_R}{S}) \doteqdot 146{,}000(1 - \frac{1280}{2960})$

$\doteqdot 83{,}000$ CFS

@ $Q_{p3} \doteqdot 83{,}000$ CFS

$H_3 \doteqdot 14.5'$

$V_R \doteqdot 970$ Ac-ft.

AVE. STORAGE IN LAKE DAWSON : $V_{AVE} = 1130$ Ac-ft.

(v) PEAK FLOOD OUTFLOW. Q_{p3}

$Q_{p3} = Q_{p1}(1 - \frac{V}{S}) \doteqdot 146{,}000(1 - \frac{1130}{2960})$

$Q_{p3} \doteqdot 90{,}000$ CFS

$H_3 = 15'$

IT IS PROBABLE THAT DAWSON LAKE DAM WILL ALSO FAIL UNDER THIS SURCHARGE (±9' ABOVE THE EMBANKMENT)

Cahn Engineers Inc. Consulting Engineers

Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND Sheet 7 of 7
Computed By D. SHEN Checked By [illegible] Date 6/2/1978
Field Book Ref. Other Refs. CE#27-531-GD Revisions

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE WATROUS WOODBRIDGE, CT

DOWNSTREAM DAM FAILURE HAZARD

(1) (CONT'D) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

(7) SUMMARY:

PEAK FAILURE OUTFLOW: $Q_{P1} = 193{,}000$ CFS

REACH OUTFLOW (U/S OF LAKE DAWSON)

$Q_{P2} = 146{,}000$ CFS

STAGE ≈ 25'

PEAK FLOOD OUTFLOW (FROM LAKE DAWSON)

$Q_{P3} \approx 90{,}000$ CFS

SURCHARGE ABOVE SPILLWAY $H_3 \approx 15'$

DAWSON DAM WILL BE OVERTOPPED BY ± 9'.

NOTE: THESE COMPUTATIONS HAVE BEEN PERFORMED BASED UPON A DAM BREACH WITH A SURCHARGED WATER SURFACE ELEVATION. IN ACCORDANCE WITH NORMAL CORPS PROCEDURES, COMPUTATIONS ARE PERFORMED BASED UPON A WATER SURFACE ELEVATION AT THE TOP OF THE DAM. A DAM BREACH WITH THE WATER SURFACE AT THE TOP OF THE DAM AND WITHOUT HEAVY DOWNSTREAM CHANNEL FLOW COULD BE MORE CRITICAL THAN A DAM BREACH WITH A SURCHARGE. THE DIFFERENCE, IN THIS CASE, IS NOT SUBSTANTIAL.

APPENDIX E

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST.	STATE	COUNTY	CONGR DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	318	NED	CT	009	03				LAKE WATROUS DAM	4123.1	7258.2	08SEP78

POPULAR NAME	NAME OF IMPOUNDMENT
	LAKE WATROUS

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	07	WEST RIVER	WOODBRIDGE	3	4000

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
RECTPG	1915	S	67	51	2800	2230	NED	N	N	N	N	22AUG78

REMARKS

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NAVIGATION LOCKS NO.	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
1	1240	U	70	2800												

OWNER	ENGINEERING BY	CONSTRUCTION BY
NEW HAVEN WATER CO	ALBERT B HILL	C M BLAKESLEE AND SONS

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NONE	NONE	NONE	

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
CAHN ENGINEERS,INC	01JUN78	PL-92-367

REMARKS